US Army Corps of Engineers

(2)

MISCELLANEOUS PAPER GL-86-38

SOFT SOIL STABILIZATION STUDY FOR WILMINGTON HARBOR SOUTH DREDGE MATERIAL DISPOSAL AREA

by

Robert M. Koerner

Drexel University
Department of Civil Engineering
Philadelphia, Pennsylvania 19104

Jack Fowler

Geotechnical Laboratory

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631, Vicksburg, Mississippi 39180-0631

and

Christopher A. Lawrence

Drexel University
Philadelphia, Pennsylvania 19104

December 1986

Final Report

Approved For Public Release; Distribution Unlimited

Prepared for US Army Engineer District, Philadelphia
Philadelphia, Pennsylvania 19104

GEOTECHNICAL LABORATORY

Unclassified
SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

Form Approved
OMB No 0704-0188
Exp Date Jun 30, 1986

A177446

1a. REPORT SECURITY CLASSIFICATION Unclassified	1b. RESTRICTIVE MARKINGS
2a. SECURITY CLASSIFICATION AUTHORITY	3. DISTRIBUTION/AVAILABILITY OF REPORT Approved for public release; distribution unlimited
2b. DECLASSIFICATION/DOWNGRADING SCHEDULE	
4. PERFORMING ORGANIZATION REPORT NUMBER(S) Miscellaneous Paper GL-86-38	5. MONITORING ORGANIZATION REPORT NUMBER(S)
6a. NAME OF PERFORMING ORGANIZATION USAEWES Geotechnical Laboratory 6b. OFFICE SYMBOL (If applicable) WESGE-ET	7a. NAME OF MONITORING ORGANIZATION
6c. ADDRESS (City, State, and ZIP Code) PO Box 631 Vicksburg, MS 39180-0631	7b. ADDRESS (City, State, and ZIP Code)
8a. NAME OF FUNDING/SPONSORING ORGANIZATION US Army Corps of Engineers 8b. OFFICE SYMBOL (If applicable)	9. PROCUREMENT INSTRUMENT IDENTIFICATION NUMBER
8c. ADDRESS (City, State, and ZIP Code) Washington, DC 20314-1000	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT NO. | PROJECT NO. | TASK NO. | WORK UNIT ACCESSION NO.

11. TITLE (Include Security Classification)
Soft Soil Stabilization Study for Wilmington Harbor South Dredge Material Disposal Area

12. PERSONAL AUTHOR(S)
Koerner, R. M., Fowler, J., Lawrence, C. A.

13a. TYPE OF REPORT	13b. TIME COVERED	14. DATE OF REPORT (Year, Month, Day)	15. PAGE COUNT
Final report	FROM 1985 TO 1986	December 1986	106

16. SUPPLEMENTARY NOTATION
Available from National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161

17. COSATI CODES		
FIELD	GROUP	SUB-GROUP

18. SUBJECT TERMS (Continue on reverse if necessary and identify by block number)
Dredged material (WES)
Soft soils (WES)
Seagirt Project (WES)
Soil stabilization (L.C.)
Geotextiles (WES)
Vertical drains (L.C.) (over)

19. ABSTRACT (Continue on reverse if necessary and identify by block number)

High-strength geotextiles coupled with polymeric vertical strip drains have essentially replaced the use of sand drains in the consolidation of soft clay deposits. Soft soils described in this report were saturated fine-grained organic silts and clays with an undrained shear strength of less than 100 lb/ft^2. These materials were typical of maintenance dredged materials that are dredged by the US Army Corps of Engineers from rivers, port facilities, and harbors. This report contains a critique of the state of the art for soil stabilization using geosynthetic materials. It describes the implementation and performance of a high-strength geotextile and vertical strip drains in an ongoing project, Seagirt Project, being constructed by the Maryland Port Authority, Baltimore, Maryland. The Seagirt Stabilization Project consisted of a 113-acre dredged material containment area that contained 18 ft of fine-grained dredged material 50 to 150 percent above the liquid limit to depths of 20 to 33 ft. This surface contained "alligator cracked crust" 3 to 12 in. deep on

(Continued)

20. DISTRIBUTION/AVAILABILITY OF ABSTRACT ☒ UNCLASSIFIED/UNLIMITED ☐ SAME AS RPT ☐ DTIC USERS	21. ABSTRACT SECURITY CLASSIFICATION Unclassified
22a. NAME OF RESPONSIBLE INDIVIDUAL	22b. TELEPHONE (Include Area Code) | 22c. OFFICE SYMBOL

DD FORM 1473, 84 MAR

83 APR edition may be used until exhausted
All other editions are obsolete

SECURITY CLASSIFICATION OF THIS PAGE
Unclassified

18. SUBJECT TERMS (Continued).

Drainage (L.C.)	Sand drains (L.C.)
Wilmington Harbor South Disposal Area (WES)	Wilmington Harbor (Del.)

19. ABSTRACT (Continued).

the ground surface allowing one to walk on most of the areas to be stabilized. This report discusses the design philosophy, construction methodology, and development of new and innovative materials that are being developed for solving complex geotechnical problems.

Rapid consolidation of soft, compressible, fine-grained soils by both radial and vertical drainage with plastic strip drains can effectively reduce the consolidation time by a factor of 10. Penetration of the plastic strip drains on 5-ft centers through a sand blanket and high performance geotextile caused minimum damage to the performance properties of the fabric. A single layer of high performance geotextile with tensile strengths above 1,000 lb/in. and a minimum thickness of sand placed directly on fabric coupled with the use of low ground pressure equipment was a key element in the success of this project.

Purpose of this report was to evaluate the performance of the geotextile at the Seagirt Project and its bearing on the proposed Corps of Engineers project in the Port of Wilmington, Delaware, entitled "Wilmington Harbor South Disposal Area," and other similar projects in the future.

PREFACE

This publication describes the design philosophy, construction methodology, and implementation and performance of a high strength geotextile and plastic strip drains for stabilization of very soft dredged soils contained in the Seagirt dredged material containment area, Baltimore, Maryland.

This investigation was performed for the US Army Engineer District, Philadelphia, Pennsylvania (NAP) by the US Army Engineer Waterways Experiment Station (WES), Vicksburg, Mississippi, during the period of Jun 85 to Jan 86.

The research study reported herein was conceived and formulated by Dr. J. Fowler of the Soil Mechanics Division (SMD) of the WES Geotechnical Laboratory (GL) and Dr. R. M. Koerner, Drexel University, Philadelphia, Pennsylvania. In addition, Drs. Fowler and Koerner performed onsite inspection and supervision of this research investigation.

Specific onsite observation and inspection activities during construction were conducted by Mr. C. Lawrence, civil engineering graduate student, Drexel University, and Mr. J. D. McKenzie and Mr. S. A. Fitzinger both Geotechnical Engineers, Geotechnical Branch, NAP. Mr. B. L. Uibel, Chief of Geotechnical Branch, NAP, was responsible for contractual activities, general technical guidance and assessment of this research effort. District Engineer of the NAP during this period was LTC Ralph V. Locurcio.

This report was written by Drs. R. M. Koerner, J. Fowler, and Mr. C. Lawrence under the General Supervision of Mr. G. B. Mitchell, Chief, Engineering Group, SMD, Mr. C. L. McAnear, Chief, SMD, and Dr. W. F. Marcuson III, Chief, GL.

COL Allen F. Grum, USA, was the previous Director of WES. COL Dwayne G. Lee, CE, is the present Commander and Director. Dr. Robert W. Whalin is Technical Director.

CONTENTS

Page

SOFT SOIL STABILIZATION STUDY FOR WILMINGTON HARBOR SOUTH DREDGE MATERIAL DISPOSAL AREA

1.0 Introduction

1.1 Overview

The use of high strength geotextiles coupled with polymeric vertical strip drains for rapid consolidation has ushered in a new era in construction on soft soils. By soft soils, it is meant soils with less than approximately 100 lb/ft^2 undrained shear strength, which heretofore were essentially impossible to build upon, at least in a timely and economic manner. These soils are generally saturated fine grained silts, clays and their mixtures (often with organic material) and are typical of river transported and/or dredged materials. With the Corps of Engineers' heavy involvement in the dredging of rivers, harbors and port facilities and disposal of the dredged material, it is understandable that they should be interested in and be at the forefront in this emerging technology.

1.2 Purpose

While the Corps has had notable successes in dealing with the disposal of soft dredged soils on soft subsoils using geotextiles (e.g., Pinto Pass, Mobile, Alabama; and Craney Island, Norfolk, Virginia), the field of geosynthetics is still rapidly developing. Design philosophy, construction methods and development of new materials are constantly arising.

For these reasons, and others, it was decided to critique the existing state-of-the-art in soft soil stabilization using geosynthetic materials. The timing was somewhat influenced by an ongoing project of this type being constructed by the Maryland Port Authority in Baltimore, Maryland (referred to as the Seagirt project) which will be critiqued near the end of this report. Hopefully, the report will have direct bearing on a proposed Corps of Engineers project in the Port of Wilmington, Delaware entitled "Wilmington Harbor South Disposal Area" and on other similar Corps of Engineers projects in the future.

1.3 Scope

The emphasis of this report is on geosynthetic materials (primarily polymeric strip drains and reinforcement geotextiles) and their interaction and influence on construction on soft, compressible fine grained soils. Upon reviewing the general problem, the necessary background information will be presented to set it in its proper context. This is essentially vertical and radial consolidation with strip drains being a focal point. A complete review of currently available strip drains is also included. Stability and reinforcement concerns using high strength geotextiles are then presented. Included in this section are fabric sewing considerations and the effects of holes (necessitated by installation of the strip drains) in the reinforcement fabric.

Installation and construction considerations are then described with major emphasis on areas of potential problems. The Maryland Port Authority's "Seagirt" project is then reviewed.

The report closes with a summary and conclusion section and a separate section on recommendations. As a preview of this recommendations section it is felt that one can currently design and build with confidence on these soft soils using geosynthetic materials. As with most new areas, however, the reliability can be improved and/or design factor of safety decreased with additional inquiry into several areas. They will be described.

2.0 Statement of Problem

2.1 Various Elements Involved

Saturated fine grained soils (typical of dredged river, port and harbor materials) suffer from being both highly compressible and very weak. Thus they cannot be used, developed or even worked upon in their in-situ state. The common expedients of (a) excavation and replacement, or (b) driving deep foundations through them are generally impractical and uneconomical on a large scale and are thus prohibitive. As a result, surcharging (to essentially squeeze the water out) is a common approach for large areas. While this will eventually solve the compressibility problem, the initial weak strength of the soil makes any construction on the site essentially impossible. The need for a ground covering geotextile, functioning as a reinforcement fabric, becomes an obvious necessity. Details (both design and construction) of such a geotextile are a major consideration in this report.

Assuming that a surcharge load can be placed without a slope stability failure when using such a geotextile, focus shifts to the time for consolidation to occur. In dealing with thick (over 10'-20') deposits of saturated silts and/or clays, times for consolidation will usually be over 5 years, and often 10 to 50 years. This situation, being generally unacceptable, calls for radial consolidation methods which form the second major consideration in this report. Focus is on polymeric strip drains, rather than the more archaic sand drains.

These two major items will be reviewed (in the context of the Seagirt project) for implementation in the Corps of Engineers future construction on soft dredged soils.

2.2 Reinforcement Fabric Comments

The fabric (note that the words "fabric" and "geotextile" will be used interchangeably) which will be mainly considered for this type of work can be classified as "high performance", versus the more common light weight geotextiles which form the bulk of the products of the geotextile industry. This is because only one layer of fabric can usually be placed at the site and the subsoils need as much help as possible. Numerical examples illustrating this feature will be given. High performance fabric (versus conventional fabric) calls for fabrics with high strength, high modulus, low elongation and low creep. High survivability properties, i.e., puncture, impact, tear and burst resistances, will also be required.

Fabric seams and seaming methods play a pivotal role in the performance of the fabric. As will be seen, the seams unfortunately form the "weak link" in the reinforcement system. The effect of holes on the reinforcement performance of the seamed fabric will be an obvious concern. Unfortunately, these holes are required for the installation of the strip drains.

2.3 Strip Drain Comments

To hasten consolidation of weak compressible soils, radial drainage is preferred over vertical drainage. Drains for this purpose were formerly made by using vertical columns of sand (called "sand drains" or "drain wells"), and now the current trend has swung heavily toward polymeric strip drains. Strip drains (also called wick drains) consist of tubular, mesh, or net drainage cores protected by geotextile sheaths surrounding them and acting as a filter. Some are completely utilized into a single system.

Design concepts for strip drains will be presented along with flow rate quantification, crush strength, and potential "kinking" and "smear" problems. The geotextile filter will also be assessed in light of typical requirements. Such strip drains will be compared to conventional sand drains by means of numerical examples.

2.4 Fabric Damage Assessment

The installation of the strip drains cannot be accomplished on the soft soils being considered without the reinforcement fabric and a drainage soil layer being placed first. The thickness of this soil layer is site specific (2' to 4') and is made using granular soil since it will eventually serve as a horizontal drainage blanket. The strip drain installation rig punches a steel lance (with the strip drain inside of it) through the drainage blanket layer and fabric and continues down to the design depth (Figure 1). At this depth the lance is removed leaving the strip drain behind. The damage to the fabric caused by this operation can be catastrophic. Considering that the installing lance is a minimum of 7" wide and the driving shoe on its end somewhat larger, the resulting hole can easily be 12" in length by approximately 6" in width. For a situation where the strength of the fabric (along with its seams) is already being challenged, the creation of holes is unfortunate. As this situation is necessary, it must and will be carefully

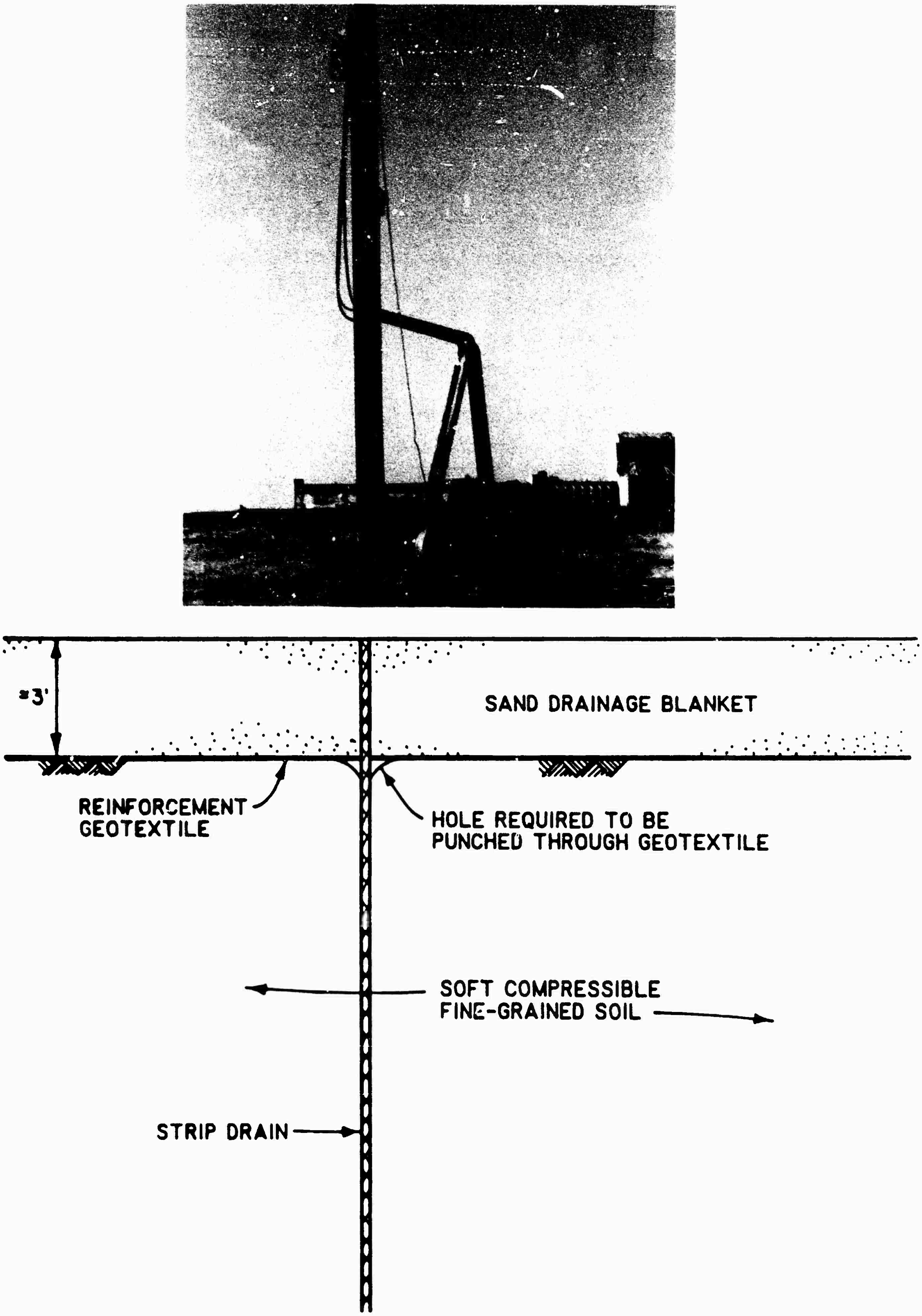


Figure 1. Typical cross section showing reinforcing fabric, strip drain, and method of installation

addressed. Numerical examples illustrating how to include such holes in the stability analysis will also be provided.

2.5 Implications to Project Design

Both of the above mentioned considerations (reinforcement fabric and strip drains) will be presented by means of an analytic approach followed by numerical examples. In this way, one could use the information via a sensitivity analysis (for a specific site) to note the optimal fabric and strip drain designs. The resulting behavior should be reflected in lower project costs and greater overall efficiencies.

2.6 Implications to Project Performance

Since building on very soft soils is extremely risky, the utmost concern must be given to design, plans, specification, monitoring and observed field performance. This is indeed a new "art", where we learn from project to project. This somewhat volatile state will no doubt continue for the near future. However, this type of construction should be used where appropriate to it since the options are not acceptable environmentally or economically and there is no risk to human life from a construction period failure. Experience shows that where construction failures have occurred, with this type of construction, the cost of remedying the problem has been small relative to the added cost of other construction methods. Somewhat numerous successes have indeed resulted. It is these successes that are the stepping-stones on which this report is based.

3.0 Background

3.1 Construction on Extremely Soft Soils

Extremely soft soils are characterized by high water content and fine grained soil, thus both high compressibility and low shear strength are to be expected. If any structural load is placed on such soils, the mass will either fail (perhaps in a rotational arc or by a translational mud wave) or settle considerably. For these reasons such soils have either been avoided entirely or worked with very slowly. In this latter category, the use of a gradually increasing surcharge fill is common; with lift thicknesses of as little as 3" per day, shear failures have been known to occur. The point being that the rate of surcharge fill placement must be linked to the soils' initial shear strength and the rate of strength increase as indicated by the dissipation of pore water pressure and corresponding increase in effective stress. The well known effective stress equation describes the process.

$$\sigma' = \sigma - u_w \tag{1}$$

where

σ' = effective stress

σ = total stress

u_w = pore water pressure

By holding the total stress (σ) constant, any decrease in pore water pressure (u_w) will result in an increase in effective stress (σ'). Furthermore, as seen in the Mohr-Coulomb failure criterion this increase results in an immediate gain in shear strength.

$$\tau = c + \sigma' \tan \phi \tag{2}$$

where

τ = soil shear strength

c = cohesion

ϕ = angle of shearing resistance

σ' = effective stress

Thus as settlement proceeds, the soil gains in shear strength allowing yet greater surcharge load to be placed. The process continues in this manner until sufficient surcharge load is in place for the equivalent permanent load. At this time the surcharge load (or a portion of it) is removed and the permanent system is built. (Usually, rebound during the time between surcharge load removal and the construction of the permanent system is not a major problem.)

3.2 Vertical Consolidation Mechanisms

As a load is applied to a saturated soil mass it is initially held by excess pore water pressure (considered to be 0% consolidation) and gradually shifts completely to effective stress (considered to be 100% consolidation) according to equation 1. Well established is that the amount and time for this consolidation process to occur are as follows[1]:

$$\Delta H = H_1 \frac{C_c}{1+e_1} \log \left(\frac{p_1+\Delta p}{p_1}\right) \qquad (3)$$

$$t = \frac{H_2^{\ 2}\ T_v}{c_v} \qquad (4)$$

where

ΔH = amount of settlement

H_1 = thickness of consolidating layer

C_c = compressive index

e_1 = initial void ratio

p_1 = σ'_v = effective vertical stress

Δp = increment of load being added

t = time for settlement to occur

H_2 = maximum drainage path length

T_v = vertical time factor

c_v = coefficient of vertical consolidation

In the above equations H_1, e_1, H_2 and p_1 are obtained from the site's geometry and initial conditions, while C_c and c_v are obtained from laboratory testing. T_v is a constant (and a function of the percent consolidation) leaving Δp as the "forcing function" which mobilizes the entire process, i.e. ΔH and t.

Theoretically, the time versus settlement curves will be parabolically shaped. However, the amount of settlement and the time for settlement to occur varies widely from soil to soil.

3.3 Time Rate of Settlement (Example Problem)

As an example of the above described process, consider the vertical consolidation of 66' of organic silty clay (OH) under a surcharge load of 20' of soil weighing 120 lb/ft^3. The laboratory determined properties of the soil are:

$$\gamma = 70\ lb/ft^3$$
$$e_1 = 0.45$$
$$C_c = 0.32$$
$$c_v = 0.005\ in^2/min$$

Substituting these values into the above equations with:

$$p_1 = 33(70) = 2210\ lb/ft^2$$
$$\Delta p = 20(120) = 2400\ lb/ft^2$$

(a) The total amount of consolidation settlement will be:

$$\Delta H = H \frac{C_c}{1+e} \log \frac{P_1+\Delta p}{P_1}$$

$$= 66(12)\ (\frac{0.32}{1+0.45})\ \log\ (\frac{2210+2400}{2210})$$

$$= 56''$$

(b) The time for 90% of this consolidation ($T_v = 0.848$) will be calculated on the basis of single (top only) and double (top and bottom) drainage.

single drainage

$$t_{90} = \frac{H_2^{\;2}\, T_v}{c_v}$$

$$= \frac{(66 \times 12)^2\,(0.848)}{0.005}\;\frac{1}{(60)(24)(365)}$$

$$t_{90} = 202 \text{ years}$$

double drainage

$$t_{90} = \frac{H_2^{\;2}\, T_v}{c_v}$$

$$= \frac{(33 \times 12)^2\,(0.848)}{0.005}\;\frac{1}{(60)(24)(365)}$$

$$t_{90} = 50.6 \text{ years}$$

Easily seen by this example (which is taken from an actual project at the Wilmington Marine Terminal very close to the intersection of the Christiana and Delaware Rivers) is that settlements can be enormous and take extremely long times to occur.

3.4 Genesis of Vertical Consolidation Techniques

A number of different techniques have been used to mobilize the required pore water pressure illustrated in the previous problem. They are shown schematically in Figure 2 and explained below.

3.4.1 Surcharging with Soil

The oldest, simplest, cheapest and most common method of mobilizing pore water pressures in saturated soils for the purpose of consolidating them is by using piles of soil. For elevations above the drainage blanket (which must be granular), almost any soil will do. Since such a surcharge must be spread in relatively thin layers and be trafficable by the equipment bringing the fill to the placement site, some type of well graded sand, silt, and/or clay combination is necessary. A considerable range in soil type is possible. As mentioned previously, the surcharge fill is added at a rate commensurate with the ability of the consolidating subsoil to dissipate the mobilized pore

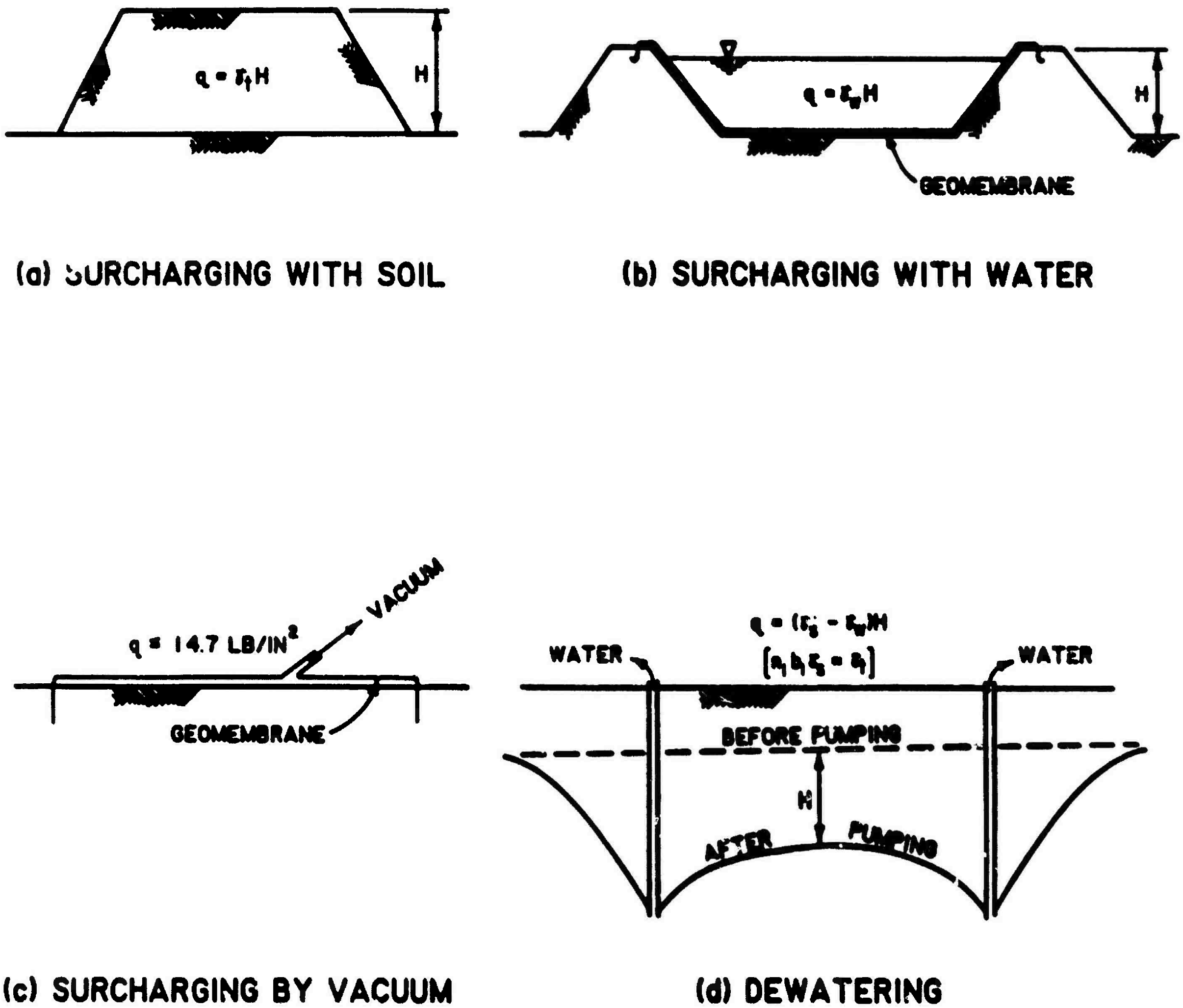


Figure 2. Various methods to apply surcharge loads to consolidate saturated fine grained soils

water pressure. This is often in the range of 3" to 12" per day, i.e. 30 to 120 lb/ft^2 per day.

Upon adequate consolidation of one zone of the site, the surcharge fill can be leap-frogged ahead to an adjacent zone depending on the project size, surcharge fill availability, and time for consolidation to occur. When the project is completed, the surcharge fill soil serves no further useful function and can be taken off site.

3.4.2 Surcharging with Water

Water placed directly on the saturated soil which is to be consolidated serves no useful function. Its only effect would be to increase the pore water pressure in direct relationship to the height of water impounded. However, if a geomembrane (or pond liner) is first placed on the ground surface and has its ends contained within an enclosing impoundment dike, water can be very effectively used to mobilize pore water pressures. This technique has been used on a number of large sites which have an ample supply of water nearby, e.g., Elizabeth, New Jersey port facilities stabilization. On that project, containment dikes were constructed in a box-like fashion, the geomembrane was placed and anchored at the top of the dikes, and then river water was used for the surcharge load. When one zone was completed, the system was moved to an adjacent one. It was very effective both technically and economically.

3.4.3 Surcharging by Vacuum

While this method is not used very often, it is possible to place a geomembrane on the ground surface, toe it in around the periphery of the site and apply a vacuum to its underside. The maximum amount of surcharge that can be theoretically mobilized is $14.7 \times 144 = 2120$ lb/ft^2 which is approximately equal to a 20' high soil surcharge. Lateral escape routes of the vacuum, however, limit the technique as does a relatively high cost of its implementation.

3.4.4 Dewatering

By lowering the watertable using vacuum wellpoints surrounding the site, the effective weight of the soil is increased by an amount equal to $(\gamma_t - \gamma)$. This amounts to about 40 to 60 lb/ft^3 which is one-half the weight of the soil. Thus a 10' dewatering system is equivalent to about 5' of soil surcharge. The method has been used where surcharge soil is unavailable or where the surcharge fill height is objectionable, e.g., at airports. It is, however, quite expensive since the dewatering pumps must be kept operating on an around-the-clock basis.

3.5 Benefits of Radial Consolidation

The results of the example problem in section 3.3 clearly show that the time for consolidation is too long for most practical situations, and that the culprit is the drainage path length "H_2" in equation #4. If this value could be decreased (note that it is squared in the relationship), then marked decreases in the time for consolidation to occur can be realized. Since it is essentially impossible to intersperse horizontal drains in deep strata, the concept of vertical columns of high drainage material was attempted. Kjellman reported of using "cardboard wicks" threaded into the soil by a "stitching machine" in a 1938 paper. Several attempts were made in Europe using this technique and the machine was brought to Canada where several more attempts were made. Severe problems were encountered, however, when the cardboard became wet. The wicks were hydraulically clamped to a lancing device and often failed by pulling apart in tension. Driving through any type of obstacle was also a problem which tore the cardboard wicks.

Emphasis shifted to vertical columns of sand installed by a closed mandrel or through a hollow stem auger. These sand drains were eagerly accepted by the geotechnical engineering profession. They were particularly helped via Barron's classic paper(2) on the theoretical concepts of radial drainage published in ASCE Transactions in 1948. This study continues today to be the basis of radial consolidation theory and design.

3.6 Radial Consolidation Mechanisms

It is of interest to compare the governing differential equations for vertical flow (Terzaghi-type) and for combined vertical and radial flow (Barron-type) and radial flow only (Barron-type). These three equations are listed in order following where c_v and c_h are the vertical and horizontal coefficients of consolidation.

vertical drainage only

$$\frac{\partial u}{\partial t} = c_v\left(\frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 u}{\partial y^2} + \frac{\partial^2 u}{\partial y^2}\right) \qquad (5)$$

vertical and radial drainage

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} + c_h \left(\frac{1}{r} \frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2} \right) \tag{6}$$

radial drainage only

$$\frac{\partial u}{\partial t} = c_h \left(\frac{1}{r} \frac{\partial u}{\partial r} + \frac{\partial^2 u}{\partial r^2} \right) \tag{7}$$

As noted earlier, equation #5 for vertical drainage only results in a time for consolidation of,

$$t = \frac{H_2^{\,2}\, T_v}{c_v} \tag{4}$$

while equation #7 for radial drainage only results in equation #8

$$t = \frac{d_e^{\,t}\, T_h}{c_h} \tag{8}$$

where d_e is the drain well spacing. Equation #6 for combined vertical and radial flow is used only for relatively thin compressible layers.

The differences in the predicted times for consolidation between radial and vertical drainage (eq. #4 vs. eq. #8) can be enormous. Not only is the drainage path length (H_2 or d_e) decreased significantly (again note that it is a squared value), but also "c_h" is often much greater than "c_v". This occurs whenever stratification of the in-situ deposit is present, as it generally is with water transported and deposited soil. The differences between the two time factors (T_v and T_h) are not very significant as Figure 3 indicates. Also note on the sketches of Figure 3 the definitions on n , d_e and d_w when using sand drains.

3.7 Time Rate of Settlement (Example Problem Continued)

To continue the example problem given in section 3.3, but now for radial drainage using <u>sand drains</u>, the horizontal coefficient of consolidation (c_h)

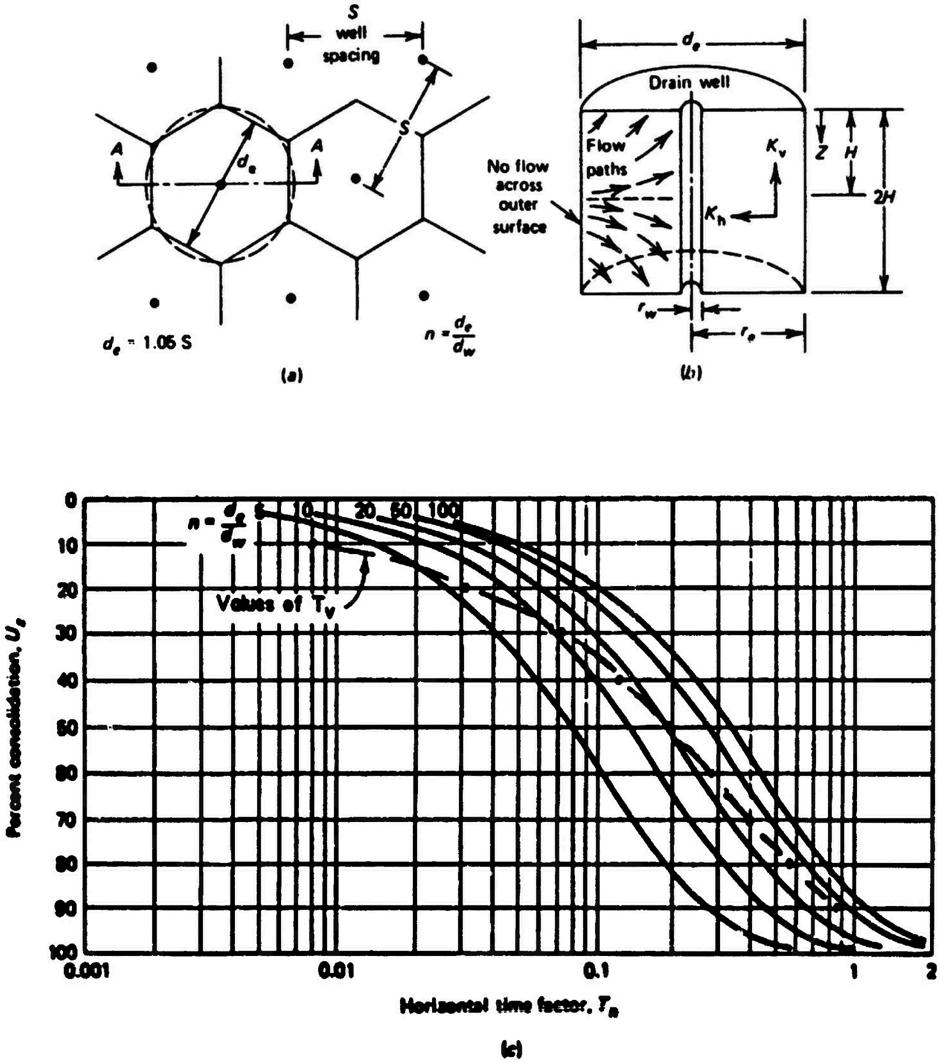


Figure 3. Theoretical results for radial consolidation to vertical drain wells. (a) Plan of drain well pattern. (b) Section A-A. (c) Values of T_h as a function of U_z for various ratios of sand drain spacing to sand drain size. (After Barron, Ref. 2)

must be obtained. This can be done in a number of ways. The simplest one is to take an undisturbed sample, rotate it 90°, and trim it to fit the laboratory consolidometer. The test is then performed in a standard manner. For the example problem to follow (which continues to be the Wilmington, Delaware marine terminal soil) $c_h = 0.010$ in^2/min will be used. This is the minimum that can be anticipated in such a soil type, i.e., $c_h = 2\ c_v$. Thus it represents a worst case scenario which results in the maximum predicted consolidation times.

For the calculations in this type of problem, it is customary to assume a given sand drain diameter and then calculate, for a series of spacings, the resulting consolidation times. Thus for d_w = 12", and d_e = 25', $n = d_e/d_w$ = 25 and from Figure 3, $T_h = 0.72$, therefore

$$t_{90} = \frac{t_h\ d_e^{\,2}}{c_h}$$

$$= \frac{(0.72)(25 \times 12)^2}{0.010} \; \frac{1}{(60)(24)(365)}$$

$$= 12.4 \text{ years}$$

For a range of spacings (and, for that matter, of different diameters if they are desired), the following table is provided. The time for consolidation can be seen to decrease drastically, e.g., for d_e = 5'; the t_{90} = 66 days.

d_w (ft)	d_e (ft)	n	T_h	r_e (ft)	t_{90} Years	t_{90} Months	t_{90} Days
1	25	25	0.72	12.5	12.4	149	4460
1	15	15	0.66	7.5	4.0	48	1400
1	10	10	0.46	5.0	1.2	14	420
1	5	5	0.27	2.5	0.18	2.2	66

3.8 Genesis of Radial Consolidation Techniques

There have been many attempts at providing for radial consolidation. These are reviewed in this section.

3.8.1 Cardboard "Wicks"

Kjellman called them "wicks". Perhaps cardboard does have some wicking ability, but clearly the drainage action is the induced pore water pressure which "pushes" the water up and/or down the vertical drainage system. As mentioned previously the cardboard wicks were not practical due to the low wet strength and inadequate installation equipment.

3.8.2 Sand Drains

Vertical columns of free draining sand (6" to 30" diameter) at 10' to 30' centers have been the "workhorse" of radial consolidation methods. Many millions of linear feet of these sand drains have been driven and the literature is repleat with references. However, there are major problems known to exist:

a. Discontinuous sand drains are sometimes created by removing the installation mandrel or auger too fast.

b. Discontinuous sand drains can be created by running out of soil in the skip supplying sand to the mandrel or auger.

c. Sand drains provide no reinforcement against a shear failure which can easily be mobilized in the soft foundation soil.

d. Some nominal amount of resistance is offered by the sand column to the escaping water.

e. The effect of side wall smear due to installation and withdrawal of the mandrel or auger is unknown and very difficult to quantify.

f. Unavailability of sand (which must be properly graded since it has to serve as its own filter) is sometimes a problem.

g. Cost (particularly of the transportation) of the sand in some locations is high.

3.8.3 Geotextile Wrapped Sand Drains

The "Chioyda Drain Pack"® method was the first to use synthetic, polymer-based, materials with the concept of radial drainage. Here geotextile "stockings" were filled with sand and driven (four at a time) within a mandrel to the desired depth. The outer steel mandrel was withdrawn, leaving the sand filled fabric stocking behind. The diameters were typically 4" to 6". This design recognized the need for a separate filter and it gave shear strength to the entire system in the form of tensile reinforcement by the geotextile filter. However, it never "caught-on" due to the introduction of newer systems not using sand at all.

3.8.4 High Transmissivity Geotextiles

This concept (originally called the Colbond® system) uses a thick needlepunched nonwoven geotextile possessing good in-plane transmissivity characteristics. The 4" wide strips are sometimes used inside a separate geotextile stocking (as a filter) and lanced into the soil to the appropriate depth. They have not been particularly successful due to their sharply decreasing flow rate at high normal pressures. Other than this limitation, however, most of the objections to sand drains listed earlier have been eliminated.

3.8.5 Polymeric Strip Drains

This new generation of materials entered the geotechnical community about five years ago and are the modernized version of Kjellman's cardboard wicks. They consist of a core material (of various configurations) protected by a geotextile filter. They have essentially revolutionalized radial consolidation by eliminating most of the objections to sand drains (perhaps not the smear) in an economic and efficient manner. They (and their many variants) are the focus of the next section.

4.0 Current Polymeric Strip Drains

While most people refer to high capacity drainage strips made from polymeric materials as "wick" drains, the term is unfortunate since there is no wicking action. It is clearly expulsion of pore water under pressure. When this pressure is no longer available, flow will stop. Thus this report will refer to these drainage strips as "strip" drains.

4.1 Types, Characteristics and Manufacturers

Probably using Kjellman's cardboard drains as the model, almost all current strip drains are about 100 mm ($\simeq$4") in width, see Figure 4. Thereafter nothing is simiiar. As seen in Table 1 following, thicknesses vary from 1.5 to 10 mm, and the cores can be made into ribs, tubes, nubs, or slots. The filter covering is usually a heat set or needled nonwoven geotextile. This geotextile acts as a true filter allowing water to pass into the central drainage core, but retaining the outside soil from piping. Such piping would, if allowed to occur, either clog the geotextile or allow soil into the core reducing or completely blocking its drainage ability. For the unitized systems with no geotextile or paper filter, the exterior of the strip drain must be perforated to allow for water inflow. However, as with the geotextile voids, these perforations are critical for they must serve two cross purposes; adequate water permittivity and soil retention.

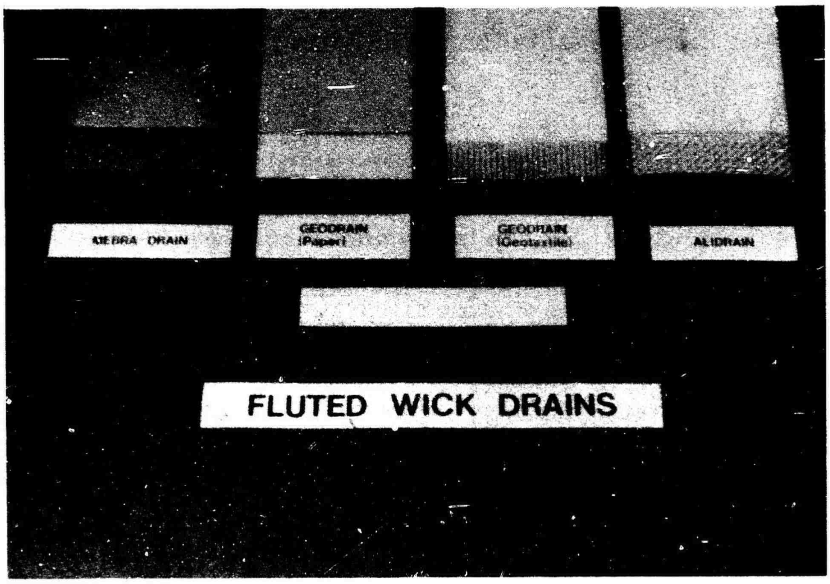
4.2 Strip Drain Spacing Design

Two different design methods will be presented in this section to determine the spacing of polymeric strip drains: the equivalent method (Koerner's) and that of Hansbo.

4.2.1 Equivalent Sand Drain Method[1]

Since much is known about the design of sand drains, one method to design strip drains is to obtain an equivalent sand drain diameter and then design accordingly. As an example problem, determine the 90% consolidation of a soil stratum with c_h = 0.010 in^2/min using Bando strip drains at varying spacings. For the solution one must measure the cross sectional area properties of a particular Bando strip drain which are;

length = 96 mm
width = 2.9 mm
void area = 92%



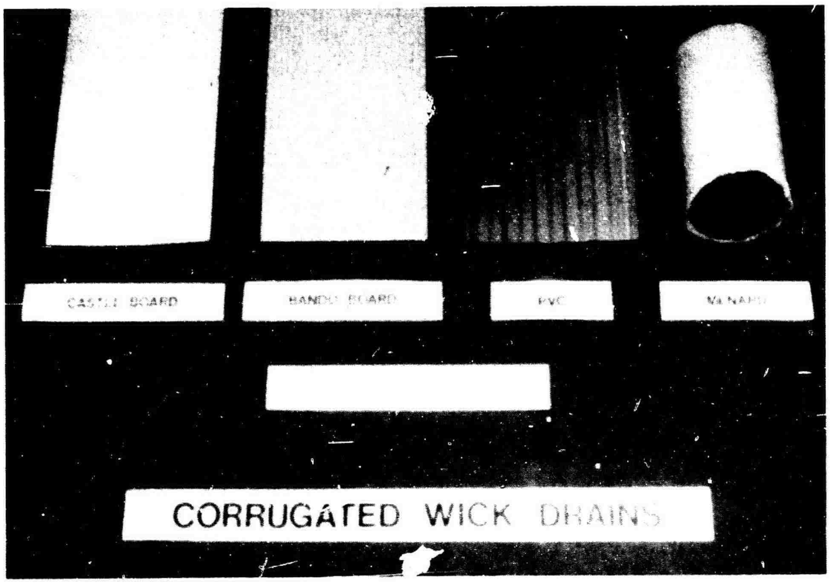


Figure 4. Various types of polymeric strip (or wick) drains

Table 1

Details of Various Commercially Available Strip (Wick) Drains

Type	Manufacturer or Sales Agent	Material	Core Characteristics Shape	No. Channels	Covering Filter	Size (in. mm)
Kjellman	SGI	paper		10	paper	100 x 3.5
Geodrain	SGI (Wager)	polyethylene		54	paper	92 x 4.0
Alidrain	Vibroflotation	plastic (?)		studs	geotextile	100 x 6.0
Castle	Japanese (Harguim)	plastic (?)		36	paper	94 x 2.6
Bando	Japanese (Fukuzawa)	PVC		48	paper	96 x 2.9
Mebra Drain	Dutch	polyethylene		38	-	-
- ester					geotextile	95 x 3.3
- pap					paper	95 x 3.2
- prop					geotextile	95 x 3.4
- sol					geotextile	95 x 2.0
Colbond	-	polyester	geotextile	∞	geotextile	100 x 6.0
PVC	-	PVC		11	unitized	100 x 1.5
Amerdrain	ICE	polypropylene		8	geotextile	92 x 10
Hitek	Burcan	polyethylene		studs	geotextile	100 x 6.0
Desol	Recosol	polyolefine		24	unitized	92 x 2.0

and calculate the equivalent void diameter of a sand drain (as an estimate use a porosity of 0.3).

- strip drain void circle diameter;

$$d_v = \sqrt{(4)(96)(2.9)(0.92)/\pi}$$

$$d_v = 18.0 \text{ mm}$$

$$= 0.71 \text{ in.}$$

- equivalent sand drain diameter;

$$d_{s.d.} = d_w = d_v/0.3$$

$$= 60 \text{ mm}$$

$$= 2.4 \text{ in.}$$

Now design proceeds as usual;

$$t_{90} = \frac{d_e^{\,2}\, T_{h90}}{c_h}$$

Using, for example, $d_e = 24''$, thus $n = d_e/d_w = 10$ and $T_h = 0.46$, so:

$$t_{90} = \frac{(24)^2(0.46)}{(0.010)(60)(24)}$$

$$= 18.4 \text{ days}$$

Continuing this process for spacings up to 90", the upper curve of Figure 5 results. This is the necessary design curve which could easily be extended for other values of percent consolidation and to other types of strip drains.

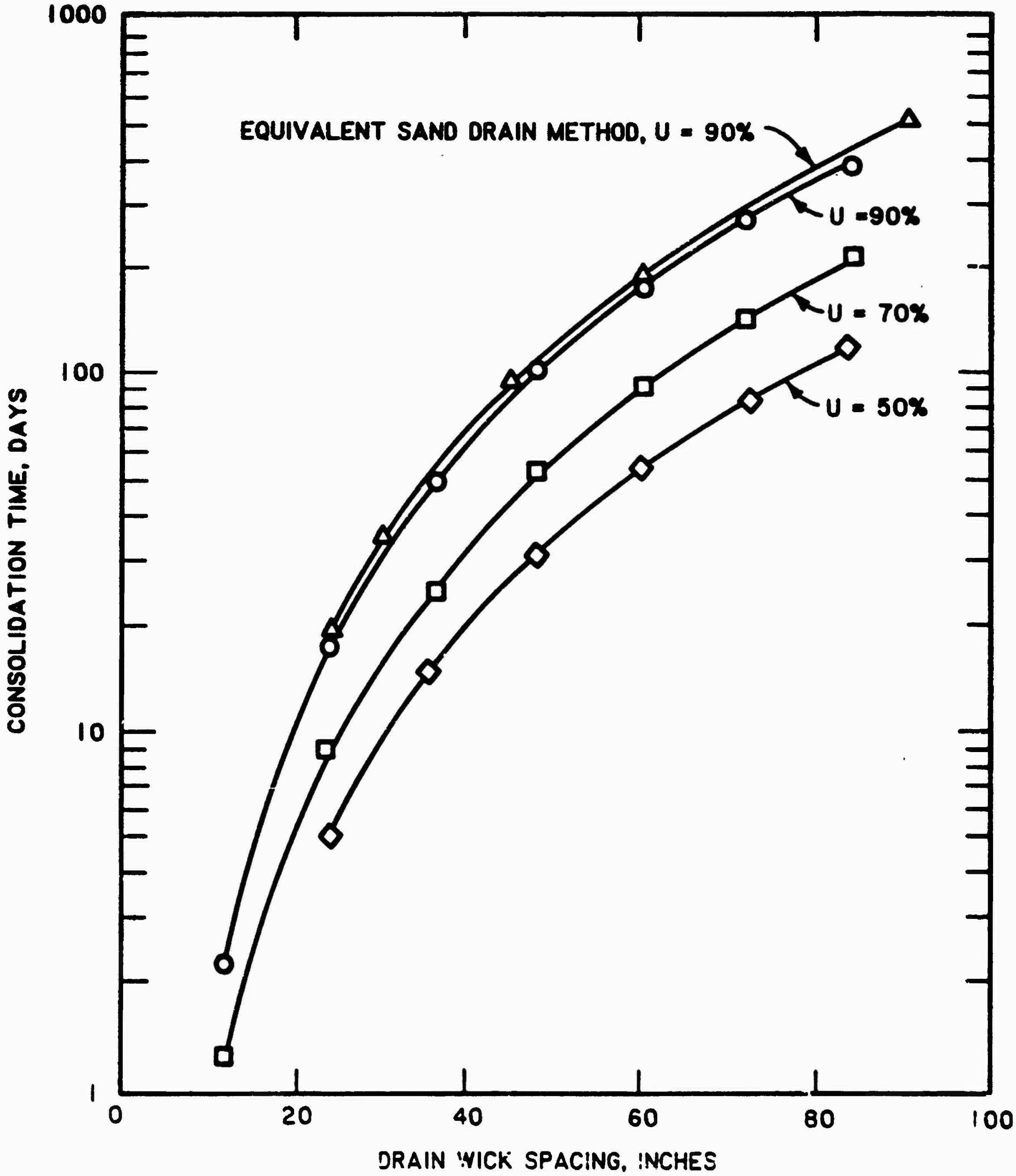


Figure 5. Results of example problem

4.2.2 Hansbo's Method[3]

The second approach toward strip drain design is more straightforward than the preceding approach and is the preferable one. As developed by Hansbo, the time for consolidation is given by the following equations which are illustrated by means of continuing the example problem:

$$t = \frac{D^2}{8c_h}\left[\frac{\ln (D/d)}{1 - (d/D)^2} - \frac{3 - (d/D)^2}{4}\right] \ln \left(\frac{1}{1 - U}\right)$$

This can be simplified, since d/D is small, to:

$$t = \frac{D^2}{8c_h} [\ln (D/d) - 0.75] \ln [\frac{1}{1 - U}]$$

where

- t = time for consolidation
- c_h = coefficient of consolidation for horizontal flow
- d = equivalent diameter of strip drain ($\simeq C/\pi$)
- C = circumference of strip drain
- D = sphere influence of the strip drain (for a triangular pattern use 1.05 spacing, for a square pattern use 1.13 spacing)
- U = average degree of consolidation

To illustrate the procedure, we continue with the previous example. Calculate the times required for 50%, 70% and 90% consolidation of a saturated clayey silt soil using strip drains at various triangular spacings. The strip drains measure 100 x 4 mm and the soil has c_h = 0.010 in^2/min.

In the above formula;

$$d = C/\pi$$

$$d = \frac{100 + 100 + 4 + 4}{\pi}$$

$$= 66.2 \text{ mm}$$

$$= 2.61 \text{ in.}$$

so

$$t = \frac{D^2}{8(.010)} [\ln (D/2.61) - 0.75] [\ln \frac{1}{1 - U}]$$

which results in the following table for consolidation times in minutes (the equivalent number of days in parentheses).

D \ U	50%	70%	90%
84"	166,000 (116)	289,000 (201)	553,000 (384)
72"	115,000 (80)	200,000 (139)	383,000 (266)
60"	74,000 (52)	129,000 (90)	247,000 (172)
48"	43,000 (30)	75,000 (52)	143,000 (100)
36"	21,000 (15)	35,000 (25)	70,000 (49)
24"	7,000 (5.1)	13,000 (8.8)	24,000 (27)
12"	970 (0.7)	1,000 (1.2)	3,000 (2.2)

These values are now plotted on Figure 5 resulting in the required design curves. Note that the D spacings must be decreased by 1.05 using a triangular drain strip pattern. When compared to the results using the equivalent sand drain method (for 90% consolidation) these values are seen to be approximately the same. The only difference is in the thickness of the strip drains used in the two examples (2.9 mm vs. 4.0 mm) which hardly matters in the above calculations.

4.3 Relevant Properties

While obviously the entire strip drain system is important, one can focus on the core separately from the filter. Concerning the core, the following aspects need careful consideration and quantification.

- o Flow rate capability of the core at the applied normal pressure it will be functioning at
- o Core breakdown pressure
- o Sustained load (creep) characteristics of the core
- o Continuity of flow if selected channels become blocked.

Concerning the geotextile filter covering of the core or the holes in the unitized body type of strip drains, the following aspects are important.

- o Sufficient void or open space to handle at least the rate of the water being expelled from the consolidating soil. This, of course, is the property called permittivity.
- o Sufficiently tight voids so that the adjacent soil is retained and will not pipe into the core. This, of course, is directly opposed to the permittivity feature and is very complex since we are dealing with cohesive soils.
- o Sufficient tensile strength (vis-à-vis, the span between contact points of the core) so that geotextile failure into the core will not occur.
- o Sufficiently low elongation and high modulus so that deformation does not markedly reduce flow in the core.
- o Adequate creep resistance so that the core flow is not reduced during the strip drain's service life.

Table 2 gives the author's assessment of the above-listed concerns regarding strip drains which are required for a confident design. As seen, there is much room for improvement in the state-of-the-art.

Table 2

Strip Drain Assessment Regarding Various Design Items

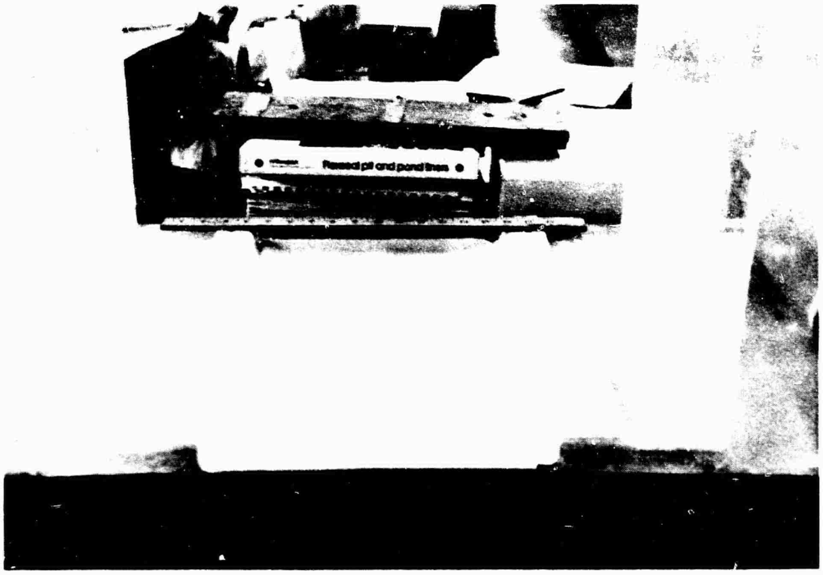
Property Under Consideration	Design Status	Experimental Status	Current Data Base
Core Properties			
Flow Rate	possible	possible	poor
Breakdown (Crush) Pressure	good	good	poor
Creep Characteristics	weak	weak	none
Flow Crossover Properties	observation	good	adequate
Filter Properties			
Flow Quantification	good	good	poor
Retention Characteristics	weak	weak	none
Tensile Strength	good	good	good
Elongation and Modulus	good	good	good
Creep Characteristics	good	good	good

4.4 Flow Rate Quantification

The flow rate capacity of strip drain cores can be measured in the laboratory. When obtained, this flow rate is compared to the water outflow of the consolidating soil to calculate a flow rate factor of safety.

The device used at Drexel University is one whereby the water flow through the core is under a constant hydraulic head while the sample is subjected to a desired applied normal pressure (Figure 6). Water is allowed to flow through the strip drain to the outlet end where it is collected and measured. The resulting hydraulic gradient can be varied from 0.06 to 2.0, but it should be recognized that strip drains function under a pressure head of varying magnitude. Applied normal pressures up to 30 lb/in.^{2} can be mobilized with this system and sustained indefinitely. This pressure is approximately that which would be developed at the base of a 100-ft-long strip drain.

The flow rate response for Amerdrain 407 (this is the strip drain being used at the MPA Seagirt project) is given in Figure 7. Since these particular strip drains are about 35 ft long, the applied normal effective pressure is approximately;



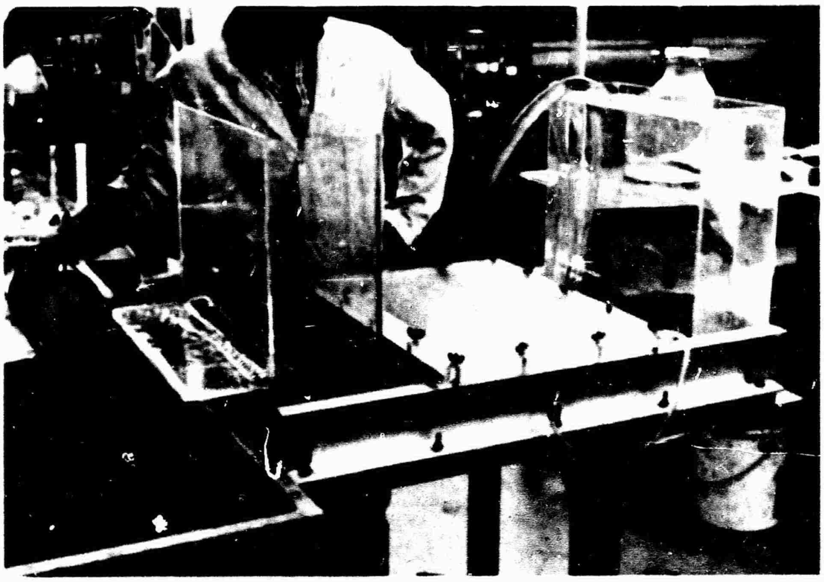


Figure 6. Drexel's device used to obtain flow rates of strip drains

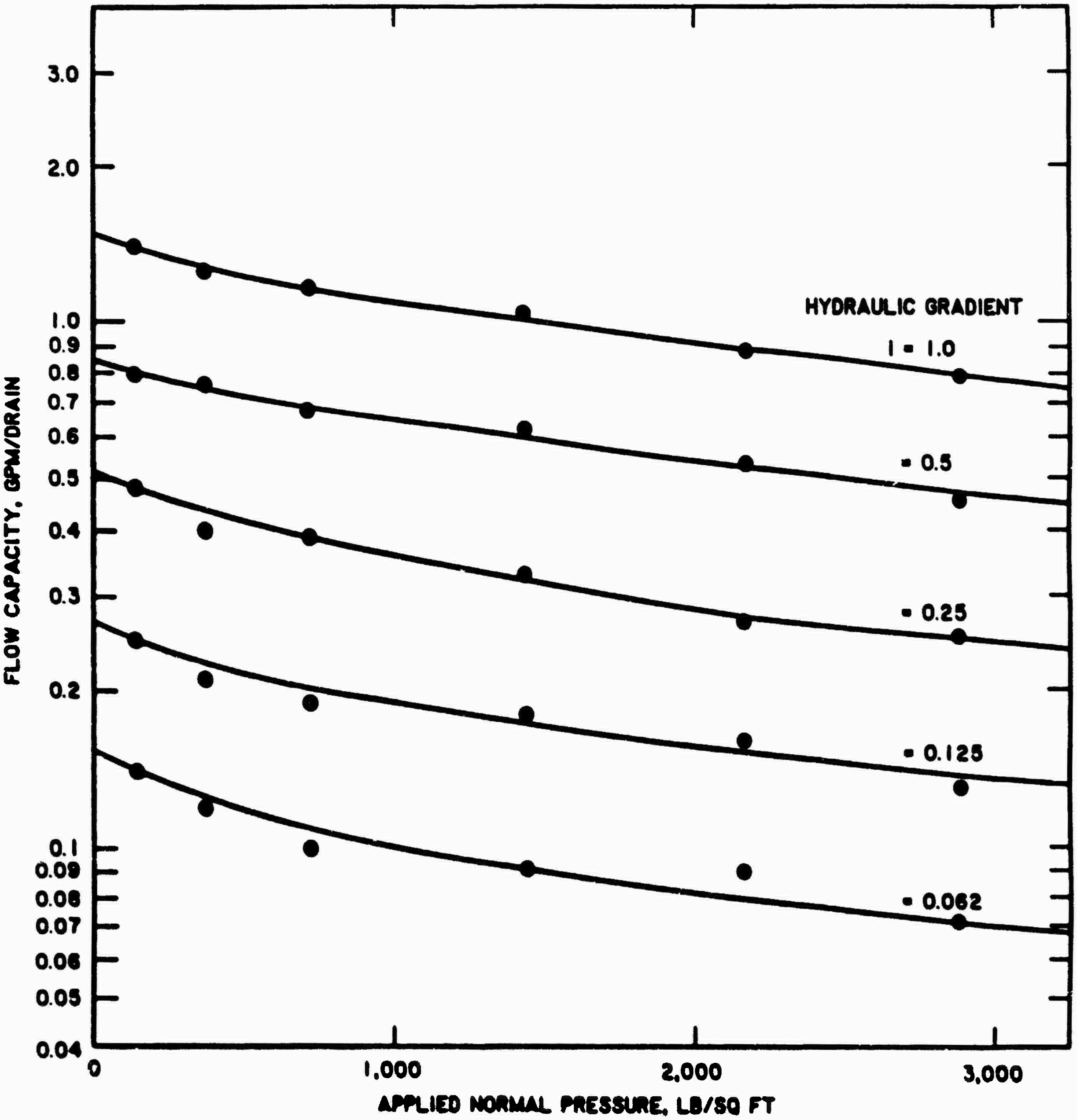


Figure 7. Flow rate behavior of Amerdrain strip drains under different gradients and pressures

$$\sigma'_h = \gamma z \, K_o$$

$$= (70)(35)(0.5)$$

$$= 1225 \text{ lb/ft}^2 = 8.5 \text{ lb/in.}^2$$

and the available flow rate at a hydraulic gradient of 1.0 is about 1.0 gal/min.

This value is now compared to the amount of water (per strip drain spacing) coming from the site during its consolidation process. For example, if the strip drains were 35' deep on 5' centers and were due to consolidate 8' within 30 days, the amount of expelled water would be equal to the following:

$$\text{flow rate} = \text{drainage area} \times \text{settlement} \div \text{time}$$

$$q_{reqd} = \frac{\pi \, (5)^2}{4} \, (8) \left(\frac{1}{30}\right)$$

$$= 5.23 \text{ ft}^3\text{/day}$$

$$= \frac{(5.23)(7.48)}{(24)(60)}$$

$$= 0.027 \text{ gal/min}$$

Thus, the factor of safety for flow rate is as follows:

$$FS = \frac{q_{available}}{q_{required}}$$

$$= \frac{1}{0.027}$$

$$FS = 37$$

4.5 Miscellaneous Concerns

There are two items related to strip drains which are not as easily quantified as was the case of the flow rate calculations in the previous section. These are "kinking" and "smear". Kinking refers to the bucking and crimping of the strip drain while its axial dimension is drastically decreased during soil consolidation. In the previous problem the axial deformation would be (8/35)(100) = 23%. If this occurred in a localized section, it could cause kinking and a potential blockage of flow. This phenomenon has been modeled by Geotechnics Holland B. V. and reported by Cortlever[4] and van deGriend[5]. Figure 8 shows the laboratory test device and a few of the deformed strip drain shapes. Even more alarming is the reduction of flow as shown in Table 3. Easily seen is that "kinking" is a real problem.

The second item considered in this section is "smear". Smear is the physical remolding of the in-situ soil as the strip drain assembly is lanced into the ground and then the empty lance removed. Since "c_h" is being used in the design, the effect this installation has on the performance of the system is not known. (This aspect of radial consolidation has a direct parallel with sand drains where the problem has been investigated for years. Unfortunately, no well defined quantification procedure has been developed.)

It is a difficult phenomenon to model in the laboratory (scale effects are horrendous), and equally difficult to assess in the field. The philosophy generally used is to keep the installation assembly as small and as "streamlined" as possible. In this way the effect should be minimized. Additionally, the spacing of the strip drains should be made somewhat closer than that required by the design formulas.

4.6 Comparison of Strip Drains to Conventional Sand Drains

Contrasting sand drains to the alternate of polymeric strip drains, a number of interesting features are revealed. The strip drains, because they consist of plastic fluted or nubbed cores which are surrounded by geotextile filters, have considerable tensile strength. Typically, the breaking strength of a 4" wide strip drain is 1000 to 3000 lbs. When threaded throughout a site on centers of 3' to 6' they offer a sizeable reinforcing effect. Furthermore, they do not require any sand to transmit flow, nor large construction equipment for installation. A rig called a "sticker" is used for installation and is relatively light weight in comparison to sand drain installation cranes. Thus the likelihood of a shear failure is somewhat reduced.

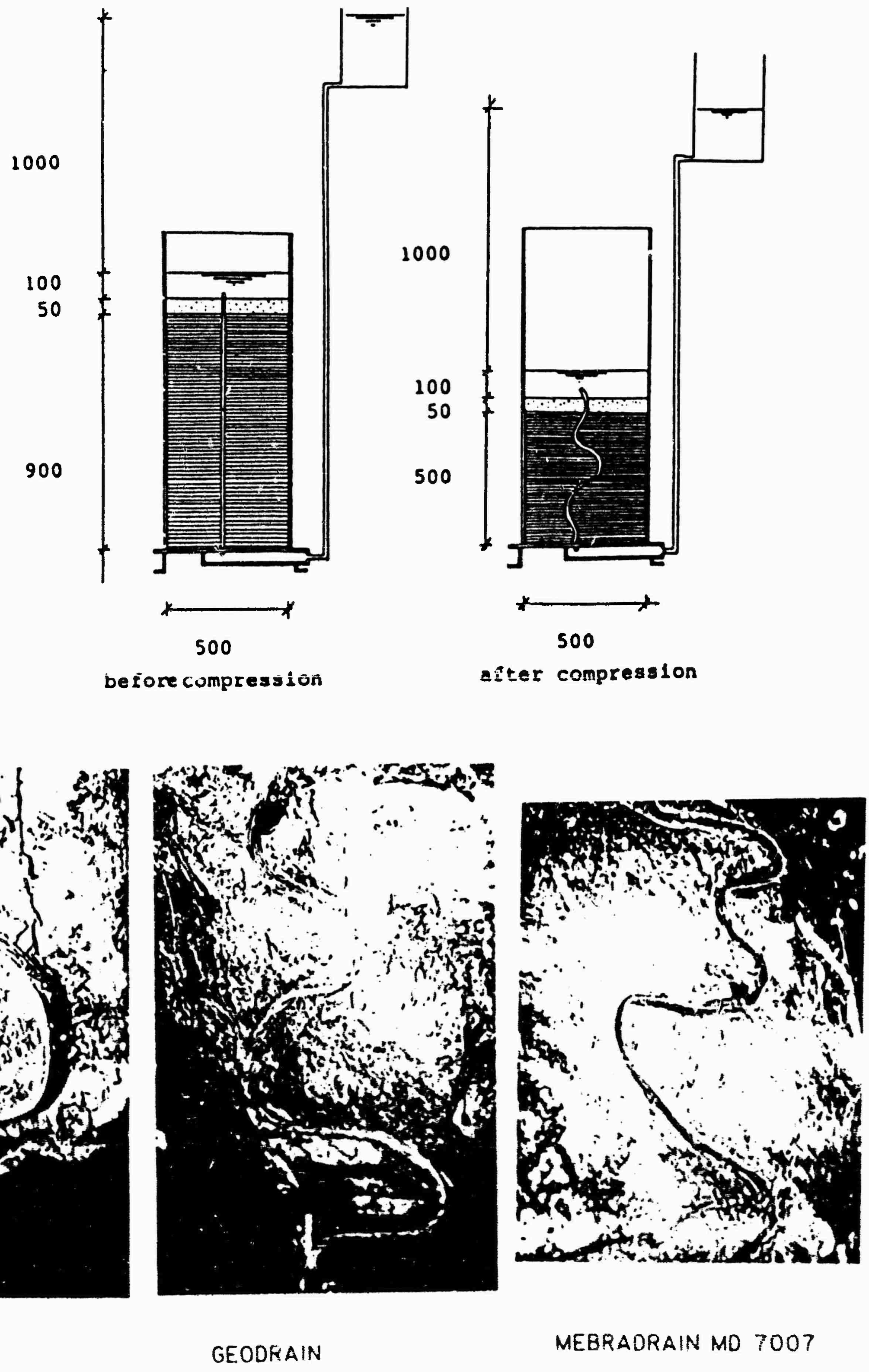


Figure 8. Deformed strip drains after compression of 44% of original heights, after Cortlever[4]

Table 3

Strip Drain Flow Rates Before and After Compression after Cortlever[4]

Strip Drain	Before Compression cc/min	Before Compression gal/min	After Compression cc/min	After Compression gal/min	Cause of Reduction
Bidim	1130	0.30	2	0.01	compression of geotextile
Desol	1908	0.50	76	0.02	buckling (kinking)
Mebra	2640	0.70	2080	0.55	negligible
Colbond	2355	0.62	1620	0.43	compression of total system
Geodrain	2160	0.57	1085	0.29	geotextile pressed into flow channels

Regarding a potential disadvantage of strip drains over sand drains, the phenomenon of "kinking" was discussed. It can be very serious and further evaluation on a site specific basis is warranted. A second problem (also occurring with sand drains) is the effect of soil smear. This includes the distortion of the soil due to installation, withdrawal and collapse of the in-situ soil on the strip drain. Its effect is mainly on the horizontal coefficient of consolidation (c_h) and it is yet to be understood. Work is ongoing in this regard under a FHWA grant to Haley and Aldrich, Inc. of Cambridge, Mass.

4.7 Current Status of Strip Drains

In summary, it is felt that strip drains offer so many advantages over sand drains that they (strip drains) will be used almost exclusively in the future. Strongly in their favor are the following items:

- Tensile strength is definitely afforded to the soft soil by installation of the strip drains. It is, however, a difficult, three-dimensional, problem to quantitatively assess.
- Properly graded sand is getting to be an expensive commodity in many areas, particularly where long transportation distances are involved.
- Unlike sand drains, there is no resistance to the flow of water once it enters the strip drain.

- o Construction equipment is generally small imparting low ground contact pressures on the soft soils when installing strip drains.
- o Installation is simple, straightforward and clean.

5.0 Stability and Reinforcement Concerns

This particular section of the report focuses on the inherent instability of the soft in-situ soils being consolidated and on the basic method to be used for their stabilization. This, of course, involves the use of a high performance fabric (geotextile) placed directly on the surface of the soft soil.

5.1 Overview

The soft soils under consideration have extremely low bearing capacities. For example for cohesive soil having 100 lb/ft^2 undrained shear strength, the bearing capacity of a long strip footing is:

$$
\begin{aligned}
q_o &= 5.7\ c \\
&= 5.7\ (50) \\
&= 285\ lb/ft^2 \\
&= 2.0\ lb/in.^2
\end{aligned}
$$

This is inadequate for anything but the lightest of low contact pressure construction equipment and just barely supports the weight of an individual. Clearly, these soft soils need strengthening and this is precisely what the geotextile is intended to provide.

5.2 Fabric Reinforcement Requirements

Placed directly upon the in-situ soil is the reinforcement fabric. Due to the soft nature of the soil it is mandatory to place the fabric with the least disturbance possible. If a surface crust exists at the site it should be kept intact as it can sometimes support laborers and sewing equipment and thereby expedite fabric placement. A single fabric layer of maximum achievable strength is the targeted geotextile. Furthermore it is necessary that seams be sewn since overlaps would be enormously large and difficult to accurately control.

5.3 Sand Drainage Blanket Considerations

A sand drainage blanket placed directly above the reinforcement fabric is necessary for a number of reasons. Its purposes are to (a) provide a working platform for the strip drain installation rig, (b) laterally drain the water which will come up from the strip drains after surcharging begins, (c) initially tension the fabric thereby mobilizing a portion of its strength and

(d) initiate the surcharge fill itself. A high permeability sand and/or gravel is necessary because of both the drainage requirement and the fact that after settlement it will be beneath the completed structure. The drainage blanket must be sloped and collector pipes may be required to remove the excess water. Usual thicknesses of the sand blanket are from 2' to 4', but this is actually a designed value which will be discussed later in the slope stability section.

5.4 Surcharge Fill Considerations

After completion of the strip drain installation (which penetrates the drainage blanket, the reinforcing fabric and the soil to be consolidated) an additional surcharge fill is applied above the sand drainage blanket. This usually consists of soil, but can be different as described in section 3.4. When soil is being used it is placed in horizontal layers until the final height is reached. The final height and degree of compaction depend upon the intended use of the site. For example, in Baltimore 600 lb/ft^2 loads are anticipated after surcharge removal so approximately 8' of soil at 115 lb/ft^3 is being used (some amount of rebound is anticipated). After completion of about 90% consolidation, the surcharge load is removed and moved to another portion of the site or off the site completely.

5.5 Slope Stability Considerations

Each of the above stages is carefully controlled so as not to create a stability failure. Since instability usually starts in the foundation as the fill is progressing it is analyzed as a slope stability base failure. There are three stages of particular concern: during placement of the drainage blanket, during strip drain installation, and immediately after (or during) surcharge fill placement. Each case will be illustrated by means of example problems. The following assumptions will be used throughout.

a. circular arc failures will be assumed to occur
b. soil strength is based on its undrained shear strength, i.e., there are no separate "c" and "ϕ" components
c. fabric strength will be used as a working stress, i.e., ultimate strength (breaking strength) divided by a suitable factor of safety 1.5.
d. fabric strength is horizontal with a moment arm vertical to the center of the slip circle (this is conservative versus the use of the slip circle's radius)

Using these assumptions, the configuration shown in Figure 9 results in the following equation for factor of safety[6].

$$FS = \frac{\tau_f R \ell_f + \tau_e R \ell_e + T_a y}{W_f x_f + W_e x_e}$$

where

FS = factor of safety

τ_f = shear strength of the foundation soil F/L^2

τ_e = shear strength of the embankment soil F/L^2

R = radius of slip circle L

ℓ_f = length of arc in foundation

ℓ_e = length of arc in embankment

T_a = allowable tensile strength of the fabric F/L

y = moment arm of the fabric L

W_f = weight of soil within the foundation failure arc R

W_e = weight of soil within the embankment failure arc R

x_f = moment arm of W_f [L]

x_e = moment arm of W_e [L]

5.6 Slope Stability (Example Problems)

As mentioned previously, three situations are included in this section. They are shown with the required data in Figures 10, 11, 12 and 13. Each is self descriptive and uses soil parameters which are common to types of situations in stabilizing dredged soils. These problems were solved on a personal computer and are available for use and extension to other related situations. The three situations which will be analyzed follow. They each have variations without, then with, a fabric reinforcement layer.

5.6.1 Sand Drainage Blanket

a. Sand blanket overlying soft soil without geotextile

b. Sand blanket overlying soft soil with geotextile

c. Sand blanket overlying soft soil with geotextile with a dozer at the edge

d. Sand blanket overlying soft soil with geotextile with a loaded dump truck at the edge

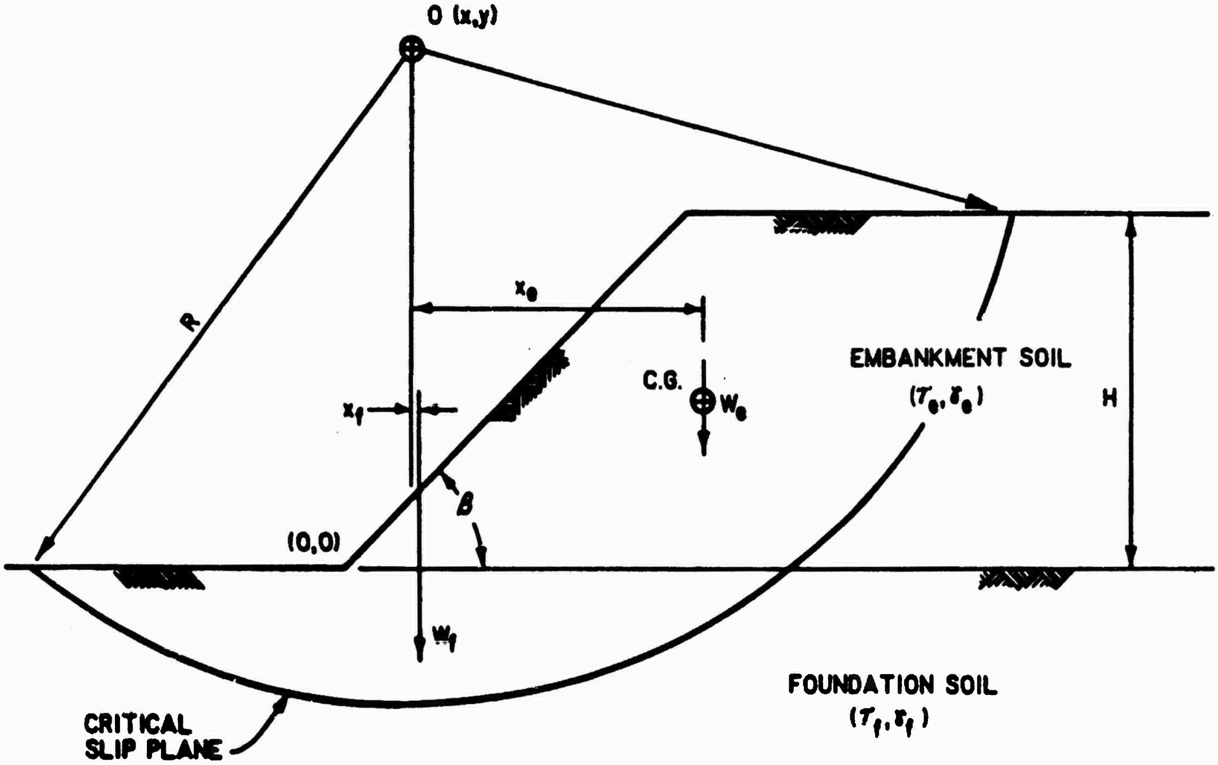
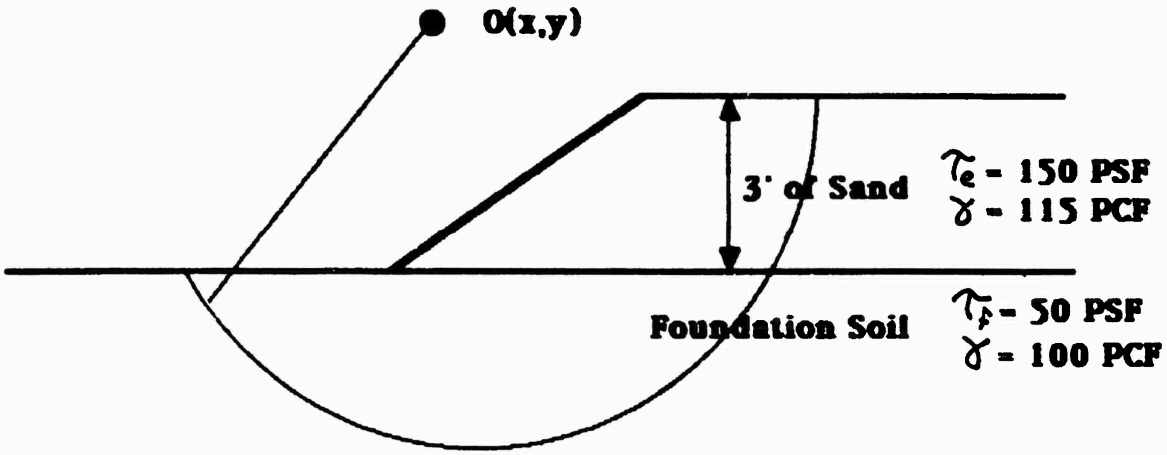
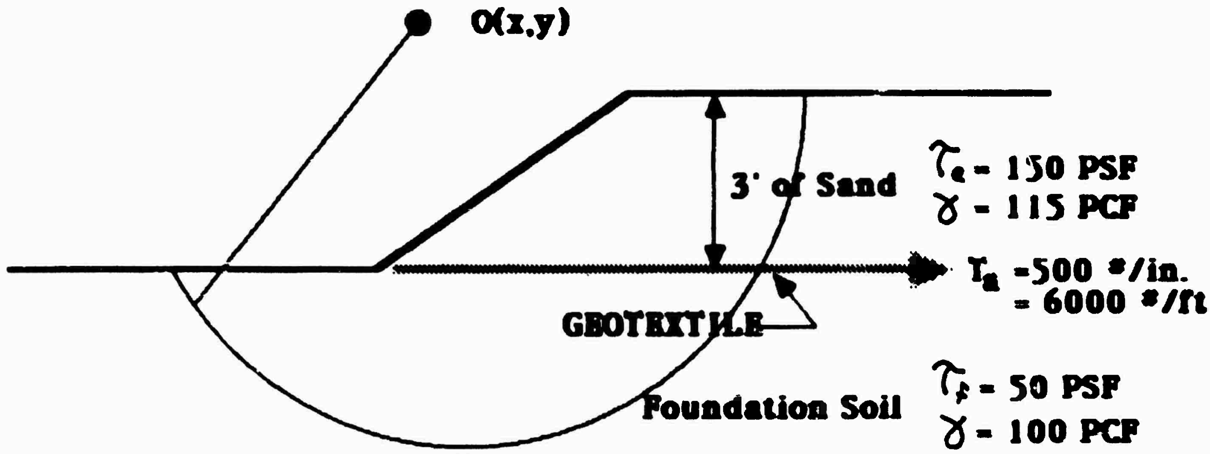


Figure 9. Cross section and nomenclature used in slope stability analyses

PROBLEM # 1



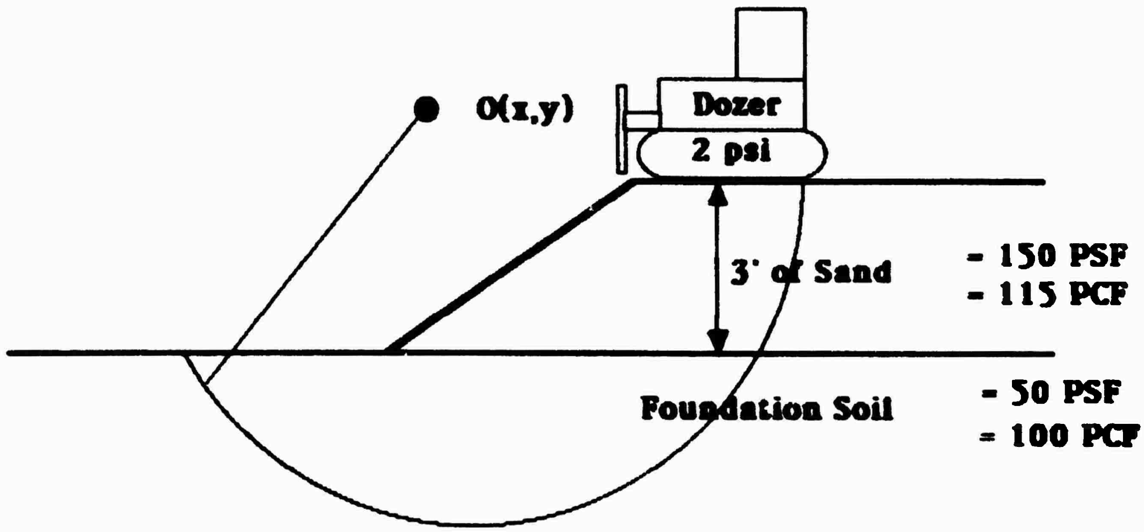
(a) WITHOUT GEOTEXTILE



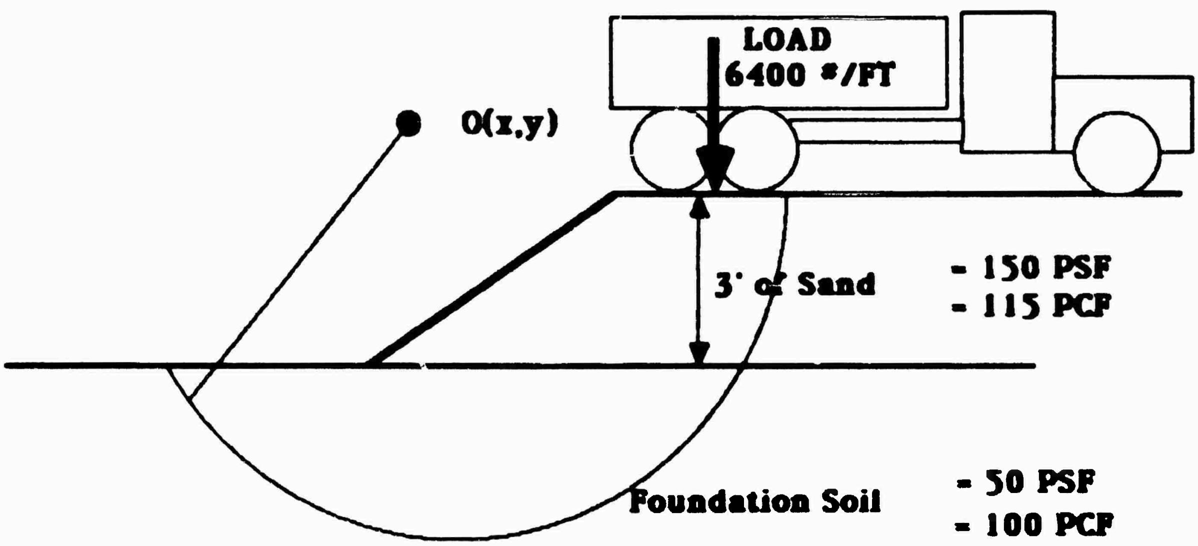
(b) WITH GEOTEXTILE

Figure 10. Stability analysis of sand blanket

PROBLEM # 1 (con't)



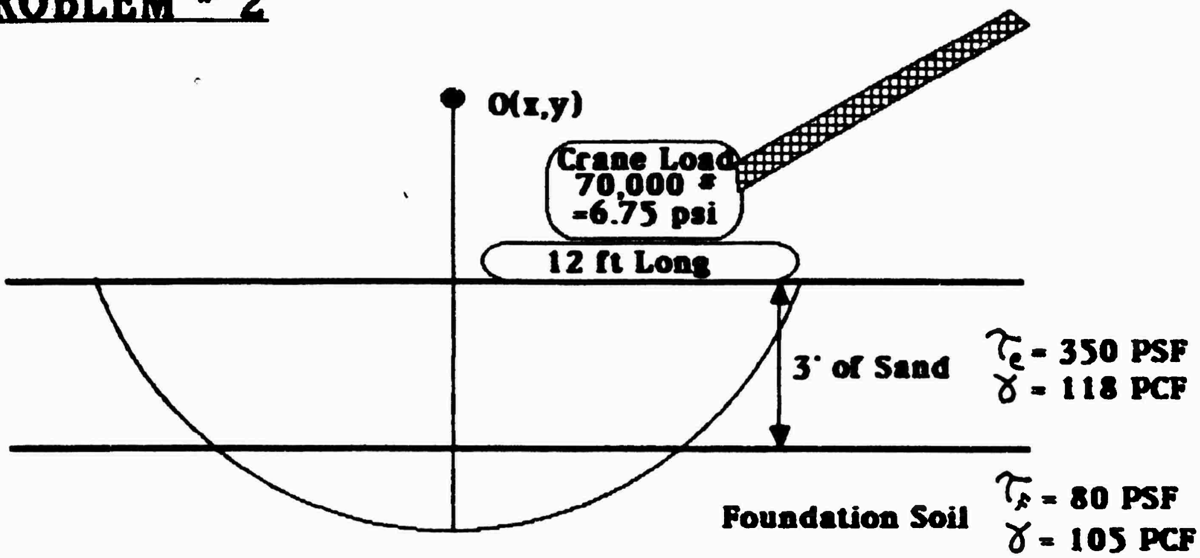
(c) WITH A LIGHT BULLDOZER



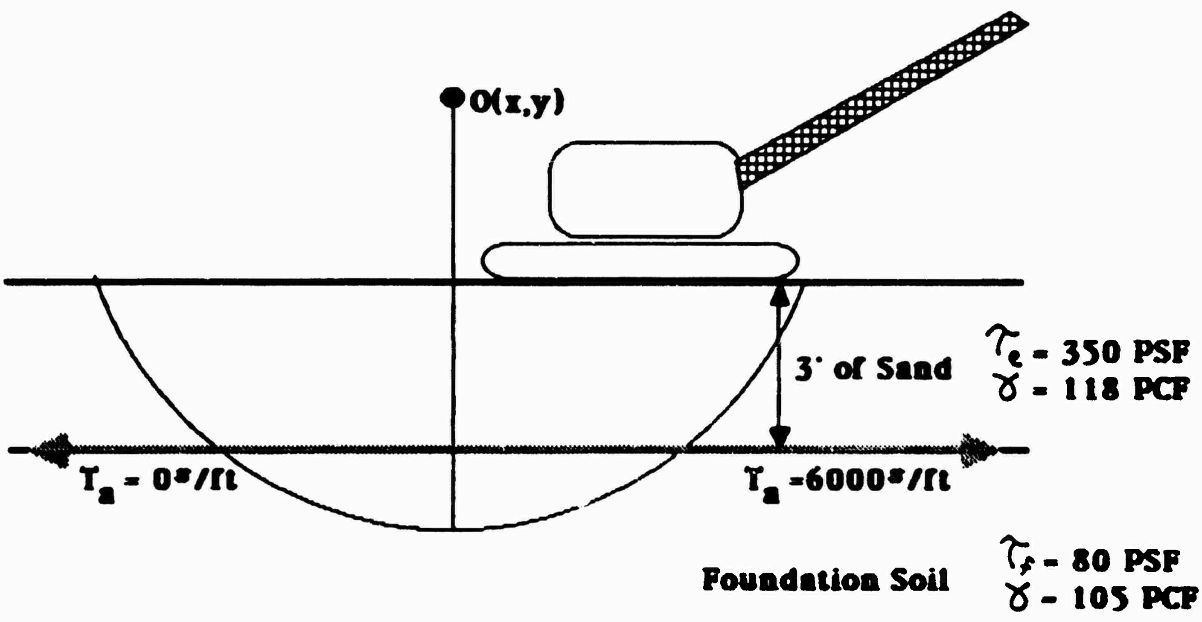
(D) WITH A FULLY LOADED DUMP TRUCK

Figure 11. Stability analysis of sand blanket (Continued)

PROBLEM # 2



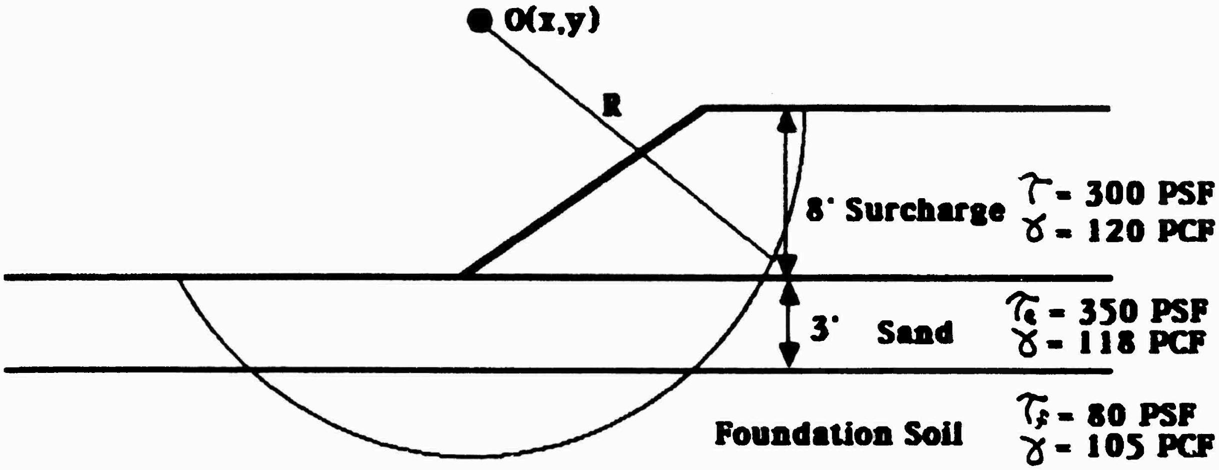
(a) WITHOUT GEOTEXTILE



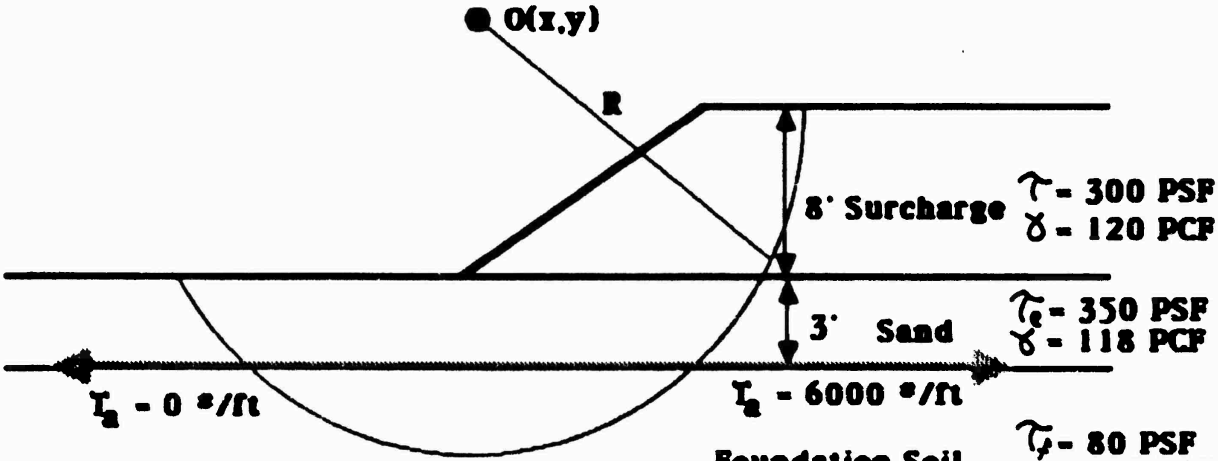
(b) WITH GEOTEXTILE

Figure 12. Stability analysis of sand blanket (Continued)

PROBLEM # 3



(a) WITHOUT GEOTEXTILE



(b) WITH GEOTEXTILE

Figure 13. Stability analysis of complete surcharge

5.6.2 Sand Blanket with Strip Drain Installation Rig

a. Crane on sand blanket overlying soft soil without geotextile

b. Crane on sand blanket overlying soft soil with geotextile

5.6.3 Sand Drainage Blanket and Surcharge Load

a. Surcharge on sand blanket overlying soft soil without geotextile

b. Surcharge on sand blanket overlying soft soil with geotextile

The sketches illustrating these situations are given as Figures 10, 11, 12 and 13, with the resulting factors of safety given in Table 4. While the data speaks for itself, the necessity of the fabric is immediately apparent, as well as the general low factors of safety under some situations. A factor of safety of less than one will result in a possible embankment failure. It is generally not desired to construct an embankment with a factor of safety less than 1.3 but embankments have been designed and constructed at 1.10. Of particular note is the case with "sand blanket and truck" which resulted in a FS = 0.8. This value, being less than 1.0, signifies that failure will occur. The teaching being that fill trucks must not be allowed to come to the edge of the fill to dump their loads. They must dump behind the edge and the (low ground contact pressure) bulldozer must push the fill over the geotextile.

5.7 Anchorage Requirements

In order to mobilize the strength of the fabric, sufficient embedment length (L_e) in the soil beyond the failure plane is required. This is considered to be the anchorage length. While some information is available, determination of anchorage length is a difficult problem and much research remains to be done. Recommendations at this time are thought to be very conservative, but this is not known for sure.

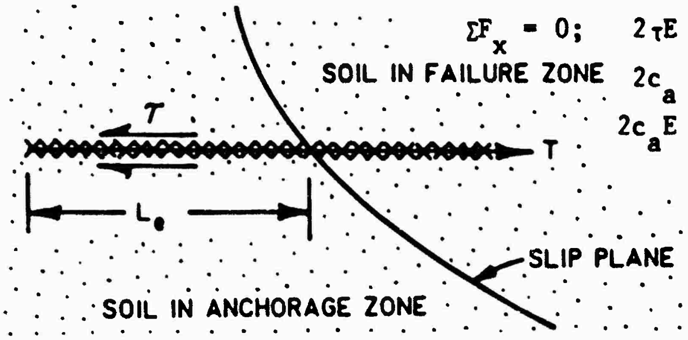
Table 4

Factor of Safety Stability Analysis Results of Problems Shown in Figures 10 to 13

Problem	No Reinforcement Geotextile	Geotextile Above Soft Soil T_{allow} = 500 lb/in.
sand blanket alone	0.8	3.4
sand blanket and dozer	0.6	2.0
sand blanket and truck	0.2	0.8
sand blanket and crane	0.8	1.1
sand blanket and surcharge	0.9	1.4

Example: What is the required anchorage length of a geotextile stressed in a soil of maximum shear resistance of 125 lb/sq ft using (E) 0.75. Use of factor of safety of 1.5.

Solution: Using the following sketch and summing forces in the X-Direction



$$\Sigma F_x = 0; \quad 2\tau E\, L_e = T$$

$$2c_a\, L_e = T$$

$$2c_a E\, L_e = T$$

$$L_e = \frac{T}{2cE}$$

$$= \frac{(250)(12)(1.5)}{(2)(125)(0.75)}$$

$$= 24'$$

where

c_a = adhesion of soil to fabric

c = cohesion of soil

$E = \frac{c_a}{c}$ = efficiency factor

This analysis assumes that adhesion is mobilized uniformly over the fabric in the anchorage zone. This is almost surely not the case. It is probably high near the slip phase and then falls off within the anchorage zone. This depends on how much movement occurs. Exactly how, remains for future research. Until then, the long lengths as computed above must be used.

5.8 Fabric Reinforcement Details

This section focuses on the specific (or certainly desirable) properties of the reinforcement fabric used between the in-situ soil and the sand drainage blanket.

5.8.1 Stress vs. Strain Characteristics:

From the results of the example problems given in section 5.6, it is seen that the in-situ soil needs strengthening. Thus on a conceptual basis the strongest fabric available should be used. This implies high strength, low elongation, high modulus and high toughness. Note that multi-layers of lower strength fabric are not recommended because of logistics and deployment problems. One layer of high performance fabric is preferred. Such fabrics are commercially available from a number of manufacturers in the 1000 lb/in. tensile strength range. While strengths greater than this can certainly be made, the load transfer across the seams becomes the weak link in the system. Sewn seam strengths of 850 lb/in. are the maximum currently evaluated at Drexel University. Thus higher strength fabric becomes excessive (and costly) since it cannot be effectively mobilized. This of course assumes that seams cannot be avoided in the particular design at hand.

5.8.2 Strain vs. time (creep) characteristics:

Strain with time under constant load can pose a problem, particularly when accompanied by stress relaxation while the system is under surcharge load. A number of analytic methods are potentially available to evaluate this situation, e.g.,

- rate process theory (using thermodynamic modeling)
- rheology (using spring and dashpot combinations)
- three element modeling

The last method is conceptually the simplest and recommended for analysis purposes. It is illustrated by means of an example problem.

Example: Given a soft clayey silt soil having experimentally determined creep properties of $m = 0.80$; $\alpha = 5$; $A = 0.0030$ %/min and a fabric having similarly obtained creep properties of $m = 0.88$; $\alpha = 3.5$; $A = 0.0015$ %/min. Determine the creep strain up to 2 years if the system is acting at 50% of its ultimate strength, i.e. $D = 0.50$.

Solution: The relevant equations are as follows, where D is 0.50, ε_1 is the initial (or elastic) strain and t is the time. Substituting the given values in these requirements results in Figure 14.

$$\varepsilon = \varepsilon_1 + \frac{A}{1-m} e^{\alpha D} (t^{1-m} - 1)$$

<u>for the soil</u>

$$\varepsilon = \varepsilon_1 + \frac{0.0030}{0.20} e^{2.5} (t^{0.2} - 1)$$

$$= \varepsilon_1 + 0.183 (t^{0.2} - 1)$$

<u>for the fabric</u>

$$\varepsilon = \varepsilon_1 + \frac{0.0015}{0.12} e^{1.75} (t^{0.12} - 1)$$

$$= \varepsilon_1 + 0.0719 (t^{0.12} - 1)$$

The key features to realize from this example are that creep strains are (a) predictable upon having the required experimental data base which must be obtained from laboratory tests (see Mitchell[10] for soil tests, Shrestha and Bell[11] for geotextile tests), and (b) they are within reason using stress levels of 50% or less of the ultimate strength of the fabric. This, in turn, should set the design mode for the fabric in that a working stress based on a

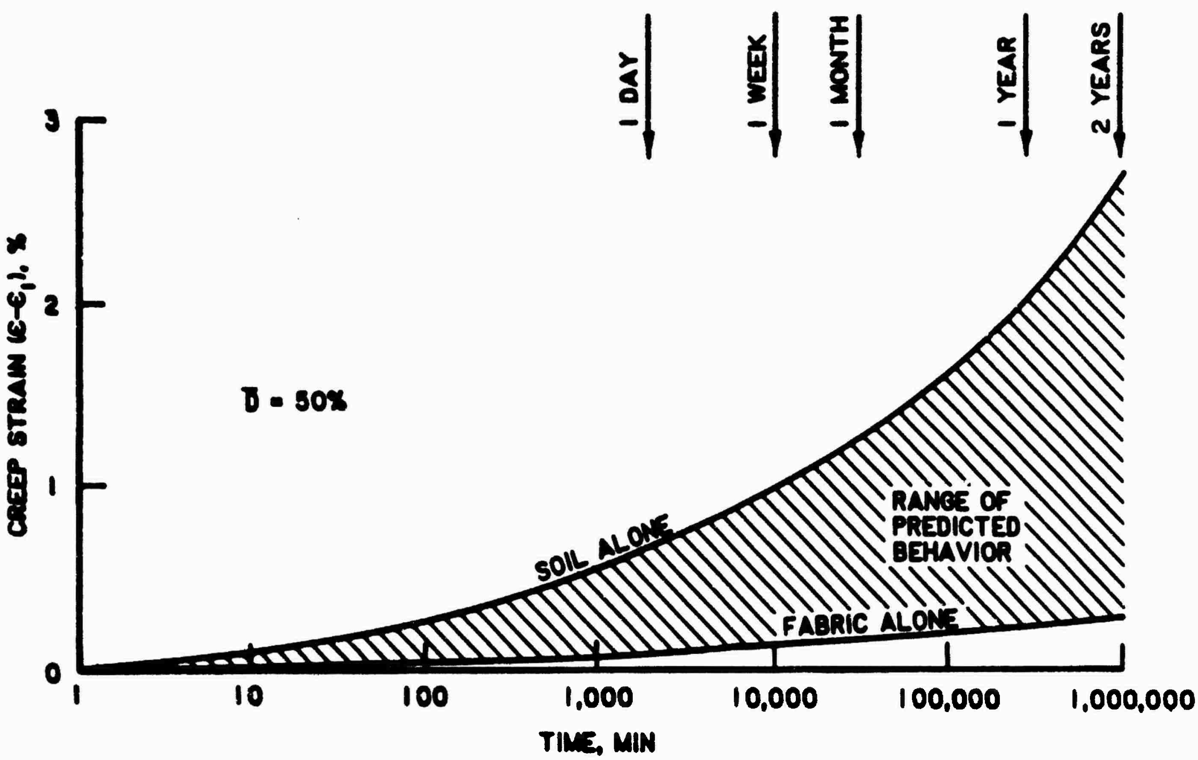


Figure 14. Results of example problem predicting creep strain according to three element model

FS $\geq$ 2.0 is required. Thus, for a 1000 lb/in. fabric, a maximum design value of 500 lb/in. should be used.

5.8.3 Survivability considerations

A number of scenarios can be envisioned where the fabric might be damaged before or during installation. Those conditions sometimes outweigh the functional design considerations presented above and are called survivability criteria. They are essentially minimum values to be used under any consideration. A number of short numeric examples are illustrated giving order-of-magnitude results for typical situations. They are adopted from Koerner[6].

5.8.3.1 Puncture Analysis - consider a 1.0" diameter stem of a bush or sapling pushing up through the fabric as the sand blanket and surcharge (11' at 100 lb/ft^3) are placed upon it. For a fabric of opening size 0.0024", the actual puncture force is:

$$T_{act} = \pi \, d_i d_a \, p'S'$$

where

d_i = diameter of fabric opening

d_a = diameter of soil particles or other object above fabric

p' = pressure

S' = shape factor

$$= \pi \, (0.0024)(1.0)(11 \times 100)(1.0)$$

$$T_{act} = 8.3 \text{ lb/in. in} \times \frac{\text{lb}}{\text{ft}^3}$$

$$T = \text{in.}^3 \, \frac{\text{lb}}{\text{ft}^3}$$

If a FS of 3.0 is used, then;

$$T_{reqd} = T_{act} \times FS$$

$$= 25 \text{ lb}$$

which is usually satisfied by high performance fabrics.

5.8.3.2 Impact Resistance - consider a 200 lb sewing machine accidentally falling directly on the fabric from a 3' height. This is an energy of 600 ft-lbs which is reduced due to the yielding of the soft soil beneath the fabric;

$$E_{reqd} = E_{max}/\text{R.F.}$$

$$= 600/21$$

$$= 28.5 \text{ ft-lbs}$$

This is also within reason of most high performance fabrics.

5.8.3.3 Tear Resistance - holes often occur in the field deployed fabric and if a vehicle subsequently runs over the area, there is a tendency to propagate a tear. The tear forces can be quite high particularly on soft subsoils beneath the fabric. While difficult to quantify, tear forces of 100 to 500 lb should be capable of being resisted.

5.8.3.4 Burst Resistance - here two situations can be envisioned; one, the fabric being pushed up into the overlying sand voids and the other created by the mud wave ballooning the fabric as sand is placed over it. The first instance should be quite low while the second could mobilize burst stresses in the 100 to 400 lb/in.^{2} range. High strength fabric should be able to handle these values.

5.8.4 Fabric Manufacture Considerations

Included in this section are general comments on fabric manufacture which will probably best suit the conditions described up to this point.

5.8.4.1 Polymer type - while polyester holds advantages over polypropylene in lower creep susceptibility, better UV resistance and higher temperature stability, polypropylene is less expensive than polyester. However, none of the above arguments are overwhelming enough to specify one over the other. Far better, is to let the free market decide on the type of polymer.

5.8.4.2 Fabric style - in comparison to woven fabrics both the non-wovens and knits suffer in their low initial modulus and limiting strength to the levels envisioned. Certainly, the multifilament woven fabrics offer distinct advantages and are recommended for this application.

5.8.4.3 Stiffness - important as far as constructability and handleability for sewing is the fabrics stiffness (or conversely, its flexibility). Treated in ASTM D-1388 it is defined as resistance of the fabric to bending. The value must be tuned to site conditions. Some of the advantages are as follows:

- o flexible is good for folding and seaming, e.g. "J-stitch"
- o stiff is good for rigidity on soft soil
- o balance is needed which is site-specific

5.8.4.4 Tear stops - As will be seen in section 5.10 relatively large holes are caused by the strip drain installation. These holes, at spacings of 5' to 20' are stress concentration points and might cause the fabric to fail between them when it is placed under high stress. To gain a degree of safety for such situations, tear stops can be manufactured into the fabric at random orientations to the warp and weft directions. As was seen, however, the seam is the critical link and here tear stops are not possible.

5.9 Sewing Considerations

A topic of major importance is the sewing of the longitudinal and transverse ends of the fabric from roll to roll. It is a critical part of the entire process and undoubtedly the weak link in the fabric system.

5.9.1 Variables involved

In considering the field sewing of geotextiles a number of details must be addressed. They are the following:

- o Thread type, where the choices are Kelvar, nylon, polyester and polypropylene (in order of decreasing strength and decreasing cost).
- o Thread tension which is usually adjusted in the field so as to be tight but to not cut the fabric.
- o Stitch density, where 2 to 4 stitches per inch are customary.
- o Stitch type, using single or double thread (double is better - single is cheaper).

- o Seam type, where a number of possibilities are available (see Figure 15), the strongest being the butterfly type.
- o Number of rows of stitches (one, two or three) as shown in Figure 15.

The sewing of geotextiles has rapidly advanced to the point where all fabric construction on soft ground should consider its use. Tensile seam strengths of 850 lb/in. have been attained and productivity has reached a point where sewing is no longer an obstacle for rapid progress of the work.

5.9.2 Longitudinal seams

These are the seams made along the long axis of two adjoining fabric sheets. They are typically made by unrolling two layers on top of one another and then seaming one side of them. Three, four or more layers can be seamed by sewing alternate sides. The connected fabric panel is then deployed transversely to cover the site. This results in the seam being on the bottom side adjacent to the in-situ subsoil. Seams can generally be made quite well using portable sewing machines (electric or air-driven) and a team of skilled laborers. since the sewing machine weighs about 100 pounds it is very helpful if a lightweight vehicle can support it and travel along with the seaming crew. This, of course, assumes that the in-situ soil can support such a vehicle. When done in this manner, the quality assurance of such seams can be quite high.

5.9.3 Transverse seams

At the long ends of the fabric sheets, the transverse edge must also be seamed. These are difficult seams to make. There is too little fabric to properly grab onto, it is held in place by the longitudinal seams and the "runs" are short, i.e. they are limited to the width of the fabric which is 10' to 15'. Furthermore, it appears almost impossible to run the transverse seam completely into the longitudinal seam at each end. Thus a gap of 6" to 12" invariably arises. Furthermore, the transverse seam looks up, and generally looks uneven and ragged. Rigid inspection is required on these particular seams.

5.10 Influence of Holes in Fabric

Of necessity, the installation of the strip drains will require punching through the fabric at whatever strip drain interval is specified. While these holes could ideally be as small as the strip drain itself (approximately 4.0"

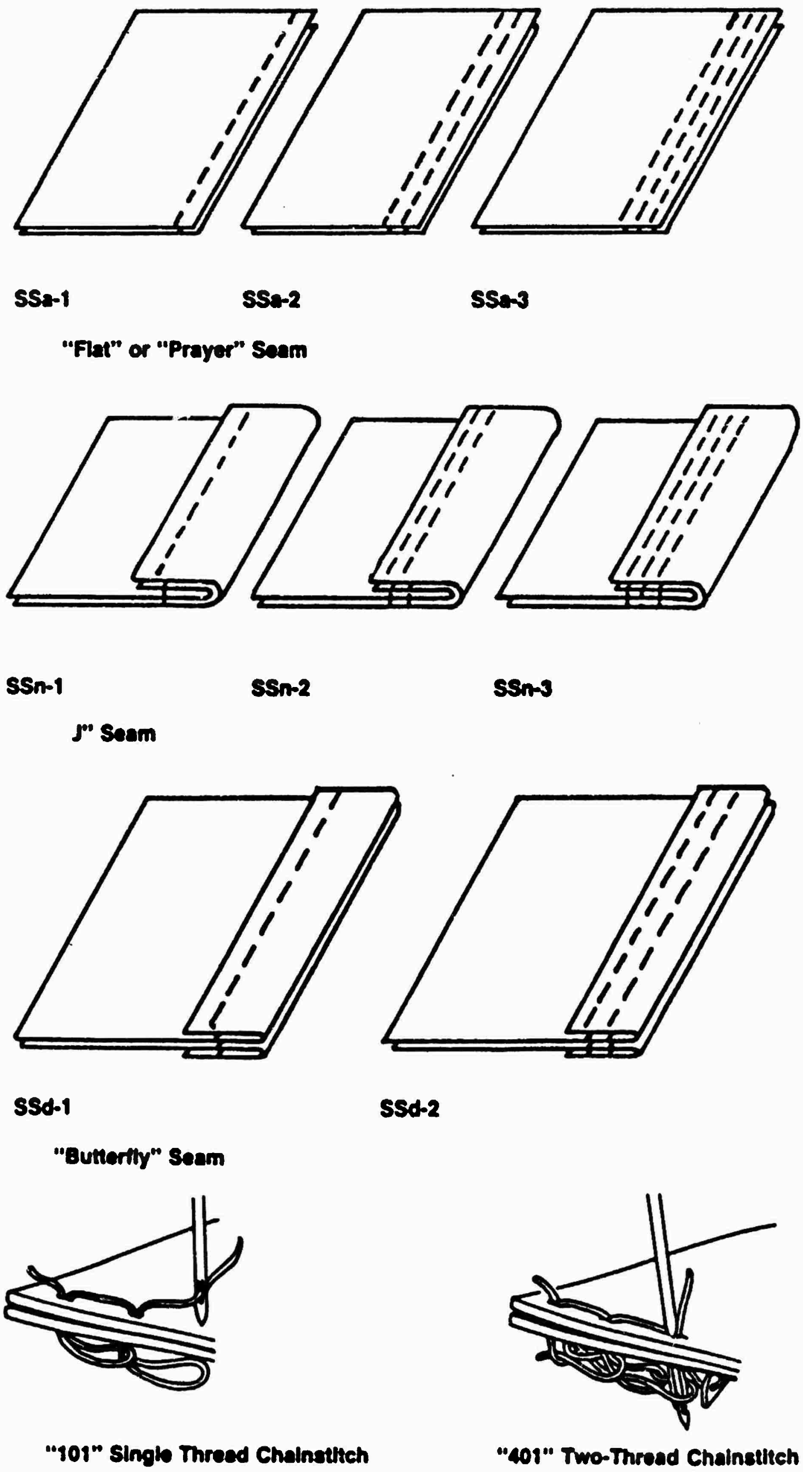


Figure 15. Various types of sewn seams for joining geotextiles

by 0.5"), the installation lance produces a much larger hole. Furthermore, the driving shoe on the end of the lance to which the strip drain is attached meets the fabric first and must be pushed through it. Different types of driving shoes will cause different sizes and shapes of holes. Typically, the holes will be 6" to 18" in dimensions and circular, elliptical, rectangular or diagonal in shape.

This section concerns itself with quantifying the effect of these holes on the fabric strength and the resulting influence on the stability of the site.

5.10.1 Fabric Strength - Recent research[7] has shown that holes in fabric (exposed as a percent reduction in cross-section) significantly decrease fabric strength. For woven fabrics, stressed in the warp, fill or bias directions, the reduction in tensile strength is approximately linear with reduction in cross section. It amounts to 0.75% strength reduction per 1.0% reduction in cross section in the warp or fill directions, and 1.8% strength reduction per 1% reduction in cross section in the bias direction. Both of these reductions are quite severe and definitely of concern. While statistically it can be challenged (due to insufficient data), the stability analysis to follow will use a 1% strength reduction for a 1% reduction in cross section. This leads to the data of Table 5 where significant fabric strength reductions can be seen for large holes at closely spaced centers.

Nowhere in the literature is mentioned the strength reduction from holes punched along the fabric's sewn seams (versus within the fabric itself). The thread's unraveling which might be created is even more critical. Hopefully, failure would not be sudden, but rather progressive as the thread yields and unravels. Research in this area should be initiated.

5.10.2 Stability - Since the strength of the fabric plays a key role in the overall site stability, its reduction due to holes must be included in the analysis. Table 5 gave a range of reductions on the basis of various spacings and hole sizes. For the example problems to follow, 12" holes at 5' strip drain spacings will be used. This amounts to a 20% reduction in fabric strength, i.e., 80% of the strength is remaining in its in-place condition.

Table 5

Fabric Strength Reduction Due to Holes

Strip Drain Spacing (ft)	Hole Size (in.)	Reduction in Fabric Cross Section (%)	Fabric Strength Remaining (%)
20	6	2.5	97.5
	12	5.0	95.0
	18	7.5	92.5
15	6	3.3	96.7
	12	6.7	93.3
	18	10.0	90.0
10	6	5.0	95.0
	12	10.0	90.0
	18	15.0	85.0
5	6	10.0	90.0
	12	20.0	80.0
	18	30.0	70.0

Example: Recalculate the stability problems worked in section 5.6 on the basis that the fabric's allowable strength is reduced from 500 lb/in. to 400 lb/in., i.e., use 80% of the intact fabric strength.

Solution: The resulting factors of safety are as follows:

Problem	No Reinforcement Geotextile	Geotextile at Full Strength T_a = 500 lb/in.	Geotextile at 80% Strength T_a = 500 lb/in.
sand blanket alone	0.8	3.4	2.8
sand blanket and dozer	0.6	2.0	1.7
sand blanket and truck	0.2	0.8	0.7
sand blanket and crane	0.8	1.1	1.0
sand blanket and surcharge	0.9	1.4	1.3

5.10.3 Regarding Drainage - With large holes in the fabric, the soft subsoils easily extrude up into the drainage blanket. The situation is analogous to a pressurized tube of toothpaste with the cap suddenly removed. The rapidly withdrawing lance aids the upwardly flow of material and leaves it deposited in a circular pile (a blob) around the in-place strip drain. The influence on the drainage capability of the sand blanket is difficult to assess, but its impact is definitely a negative one. It should be evaluated in greater detail.

6.0 Strip Drain Installation Considerations

This section is presented to describe the variations used to install strip drains. It is intended to preview the next section which is specifically oriented toward a single project.

6.1 Installation Lance

The strip drain is threaded into a metal lance for its insertion into the soil to be consolidated. The lance is generally made from steel, but it can also be of aluminum to lighten the load the installation equipment places on the foundation. This load is a major overturning force and a definite stability consideration. Its internal size must be larger than the strip drain itself with at least 1" clearance on all sides. Thus a 6" by 2.5" rectangular section would be adequate. For greater structural stiffness, however, the cross sections of the lances used are usually diamond-shaped with the long dimensions 6" to 8" and the short dimension 3" to 5".

Some earlier model lances were solid bars with the strip drain hydraulically clamped to the bottom of them. After insertion, the clamp was disengaged and the lance withdrawn. In hard driving, however, the strip drains had a tendency to tear and this type of lance was discontinued in favor of the hollow stem lance.

6.2 Type of Shoe

The strip drain only fills part of the hollow core and the soft soil may easily get pushed up into the remaining open space. The danger here is that the strip drain does not release from the lance at its intended depth due to wedged soil within the lance. To avoid this, the strip drain must be connected to an expandable driving shoe which covers the open area of the lance. Figure 16 shows sketches of a number of driving shoes. In particular note the reinforcing bar which is the simplest and least expensive method. It, unfortunately, does not cover the entire bottom of the lance and allows the soft soil to enter it. This type of connection is <u>not</u> recommended for use in soils of unconfined compression strength of less than 200 lb/ft^2. One of the other types shown which cover the entire bottom of the lance is recommended.

For penetrating stiff layers or puncturing high strength fabrics, it is necessary to have a pointed end on the bottom of the shoe. Advantages are (a) the hole size in the fabric will be minimized thereby minimizing the strength decrease in the fabric, and (b) the installation force for

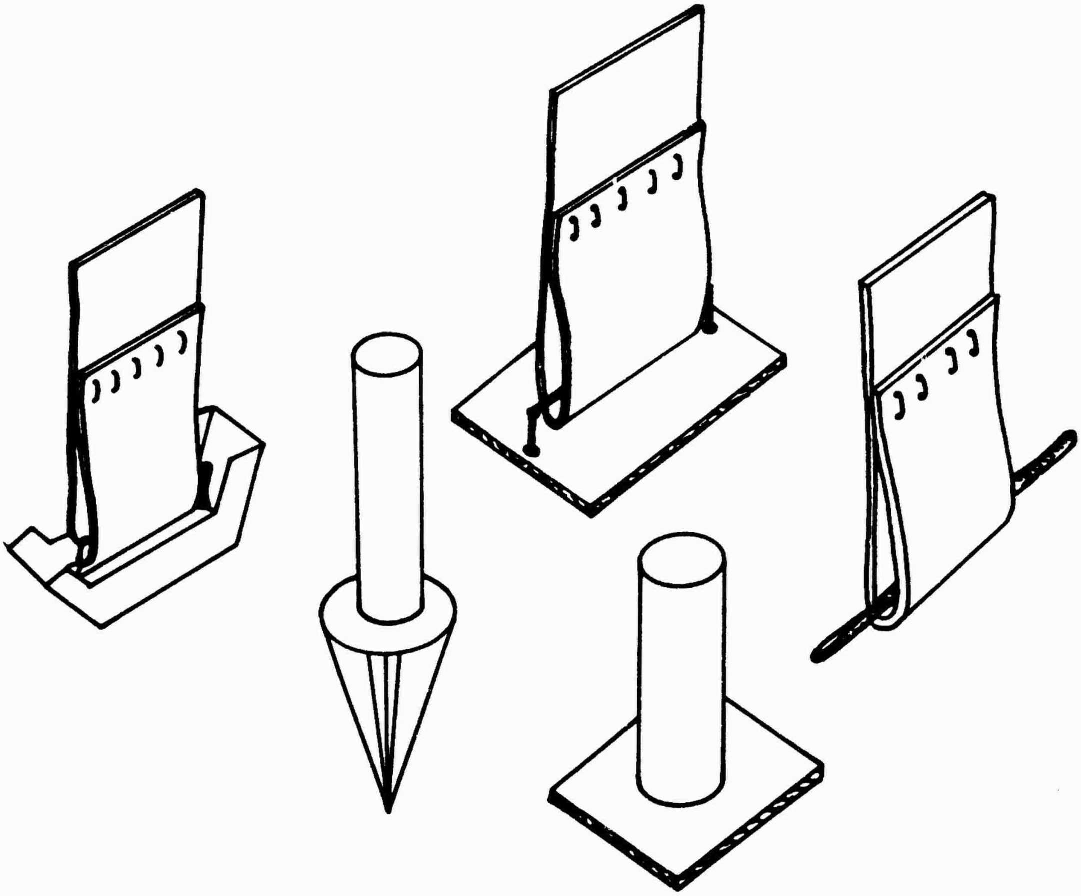


Figure 16. Various types of driving shoes used during strip drain installation

penetration will be decreased thus a lighter installation rig can be used. Both of these features are very important and should easily offset the increased cost of a pointed driving shoe.

6.3 Effect of Smear

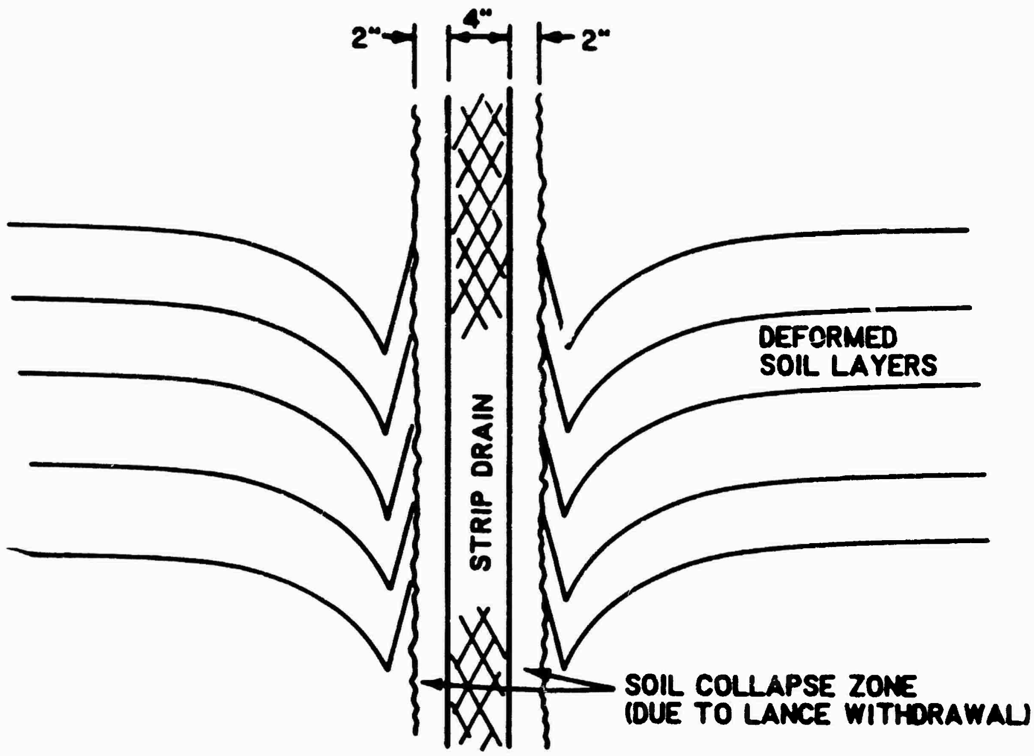
Whatever the configuration of the installation lance and the type of driving shoe, the effect on the in-situ soil is to smear it downward during lance insertion and then upward during withdrawal. The net effect is illustrated in Figure 17. Here it is seen that the influenced zone might be 4" to 6" beyond the strip drain itself. How the escaping pore water "fights its way" into the strip drain through this zone is of major (and completely unanswered) concern. The design parameter most seriously affected is the horizontal coefficient of consolidation (c_h) as seen in Section 4.2. No method is known as to how to handle this situation except to "guess" at a reduction factor for "c_h" which would result in an increased time for consolidation. In varved clays where the silt layers have orders of magnitude greater than the adjacent clay layers, this effect is more serious than in a more homogeneous material. Whether this reduction factor on c_h is 2, 5, 10 or more is simply not known and awaits further research.

6.4 Type of "Driving Hammer"

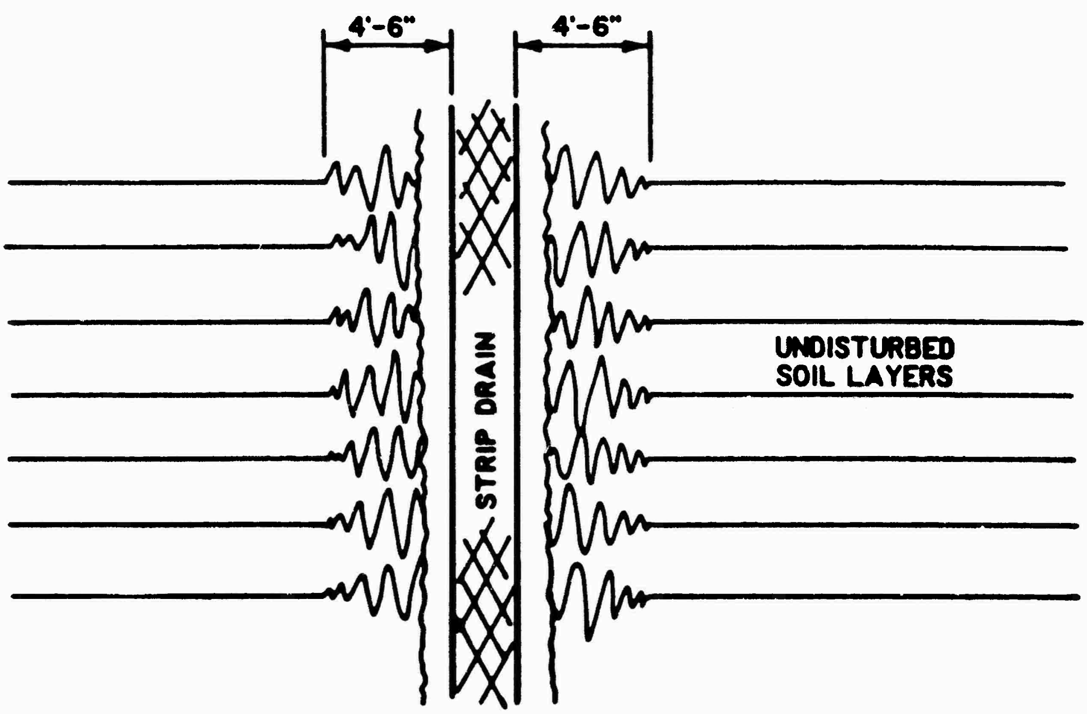
The lance can usually be inserted into the soft soil by its static weight and/or hydraulic pressure. When inserting it through stiff upper soils or through high strength geotextiles, however, pile driving methods must be used. Both single and double acting impact hammers and vibratory methods have been used. It should be noted that the forces required are generally far less than with conventional pile driving projects. Energies of 2,000 to 10,000 ft-lbs are generally sufficient versus greater than 15,000 ft-lbs for driving of conventional piles.

6.5 Type of Rig

Most rigs used for strip drain installation are crawler mounted cranes with wide tracks exerting relatively low ground pressures. This usually amounts to 5 to 10 lb/in.^{2} while such rigs are semi-stable when walking on a 2'-4' thick sand blanket supported by a reinforcement fabric, they are very unstable during the actual driving of the strip drain. It is certainly desirable to minimize ground contact pressure and the overhanging lance/hammer weight.



(a) IDEALIZED SMEAR CONFIGURATION



(b) PROBABLE SMEAR ZONE

Figure 17. Smear effects on soil due to strip drain installation

6.6 Special Equipment

The strip drain threading system can be via a roll on top of the lance or via a set of sheaves from ground level up to the top of the lance (see Figure 20). Using either method, it is then threaded down through the lance and connected to the driving shoe. Each system is slightly differently and usually trouble-free. Problems can arise, however, with weak tensile strength strip drains which tear apart requiring splicing and rethreading. Choice of equipment is best left to the contractor.

The method used to splice strip drains is dependent on the type of drain used, recall Table 1. For cores separate from the geotextile filter, they can be stapled together and then the geotextile covered back over the core. For unitized strip drains, the entire system is stapled together in a 6" to 12" overlap joint fashion. The influence of this connection on the flow rate has never been quantified. It is of sufficient concern, such that the contractor who wishes to install the unitized strip drains should prove experimentally that flow can occur across the joint. If he cannot, no joints should be allowed and continuous drains must be used throughout the project.

6.7 Sequence of Operations

The installation of an individual strip drain is quite straightforward and installation cycle times of 1 to 3 minutes should be capable of being realized for lengths of 50' or less. Of greater concern, performance-wise, is the orientation and sequencing of the work. There seems to be no procedure specified other than depth and spacing. The orientation of the strip drain seems to be at the contractor's convenience, as is the sequence of operation. While orientation might not prove to be a problem, it seems as though a systematic procedure of advancing one or two rows at a time across the site should be followed. Furthermore, it should be staged to follow the fabric installation and sand blanket placement, and precede the surcharge filling operations. Certainly, a random zone-by-zone installation procedure should be avoided.

7.0 Maryland Port Authority's "Seagirt" Project

To more specifically illustrate the stabilization of soft dredged soil using a reinforcing fabric and strip drains, the Maryland Port Authority's (MPA) "Seagirt" project will be described.

7.1 Overview

The Seagirt stabilization project is a 113 acre disposal area for dredged material from the site of the recently opened I-95 tunnel under Baltimore Harbor. The site is shown in Figure 18 and is being prepared for an extension to an existing container facility. The MPA contractor is C. J. Langenfelder and Son, Inc. of Baltimore for $10.9 million on a 600 calendar day fast-track schedule. The existing site contains 20' to 35' of dredged material (slurry) consisting predominantly of "very soft silts mixed with variable amount of clay and fine sand". It was placed 6 years before the beginning of this project. Its SPT resistance is generally a "push" or "weight of rod", i.e., its strength is too low to be measured by conventional methods. The existing water content (50% to 130%) is typically 50% to 150% above the liquid limit. The site is contained by a roadway on the inland sides and a cellular sheet pile bulkhead on the water sides. An "alligator cracked crust" 3"-12" deep exists on the ground surface, allowing one to walk over most of the areas to be stabilized.

The design consultants are STV/Lyon Assoc., Inc. of Baltimore, Maryland. The overall goal is to ready the site for an extension of the existing containerized staging area (loading, parking and unloading of containerized truck trailers) with anticipated ground surface loads of approximately 600 lb/ft^2.

7.2 Fabric Type, Seams and Design

The high performance geotextile being used is Nicolon's 62809 woven fabric consisting of multifilaments of UV stabilized polypropylene in the warp direction and polyester in the fill direction. Its mass per unit area is 30 oz/yd^2 and has a wide width tensile strength of 1100 lb/in. and 1300 lb/in. in the warp and fill directions, respectively. The fabric is shipped in 1200 lb rolls which are 16.5' wide and 270' long. It is deployed at the site by wide track dozers and shifted into position by a team of 8 to 12 laborers. Fortunately, the soil surface crust supports this type of activity.

Seaming is done using a heavy duty electric sewing machine hung on a small farm tractor. The sewing machine is powered by a portable generator

Figure 18. Aerial view of MPA's "Seagirt" stabilization project

mounted on the rear of the tractor. Again the crust of the upper surface of the dredged spoil is an advantage for the tractor and seaming crew can work directly on it. This avoids working on the previously placed fabric which sometimes can feel like a "waterbed".

The longitudinal joints are double 'J-stitched' wherein two fabric layers are lapped, folded over and the resulting four thicknesses of material (about 1/3") are stitched together. This seam is designated SSn-2 in Figure 15. Originally polyester thread was used, but the job was finished using Kevlar thread. The transverse joints are double "prayer-stitched" between ends of abutting fabric rolls. This is designated SSa-2 in Figure 15. The field crew consists of 6 to 8 laborers who shape and fold the fabric seams for the sewing machine operator.

The fabric design method used was the method presented in this report (according to the design engineer in charge), but details are not known. The specifications call for the fabric to have a 1010 lb/in. minimum strength in both the warp and the fill directions, a modulus at 10% elongation in the warp-direction of 2780 lb/in., an elongation at failure between 15% and 35%, a soil to fabric friction angle of 30°, a stiffness of 30,000 mg cm and an EOS of 0.0165" max (i.e., 0.0042 mm or << #400 sieve). It can be made from polypropylene or polyester, with polypropylene being UV stabilized. Furthermore, the minimum unseamed width is 12' and the seams must be capable of 600 lb/in. Samples for testing must be taken at each 25,000 sq yds, or less, and tested by an independent testing laboratory.

7.3 Drainage Blanket Details

Approximately 250,000 cu yds of drainage sand is placed over the fabric in a single lift which is 2.5' thick. The specifications call for the following gradation:

Sieve Size (U.S. Std.)	Percent Passing (by Weight)
3/8"	100
#4	95-100
#10	70-100
#40	15-65
#100	0-32
#200	0-15

The sand is truck hauled in from off-site and spread by two Caterpillar D3B's and one Komatsu D31P wide track dozers. Specifications call for contact pressures of these small dozers not to exceed 3.0 lb/in.^{2}

This sand layer is capped with 6" of crushed slag to provide a stabilized working surface over the sand blanket. Specifications call for either crushed rock or slag meeting AASHTO M43, size number 57 as shown in Table 903 of the MSHA Specifications. It is placed similarly to the sand and is leveled by the small dozers dragging their blades to provide access for trucks and cars, and it also was intended to keep down wind-blown sand over this large area.

7.4 Strip Drain Type and Design

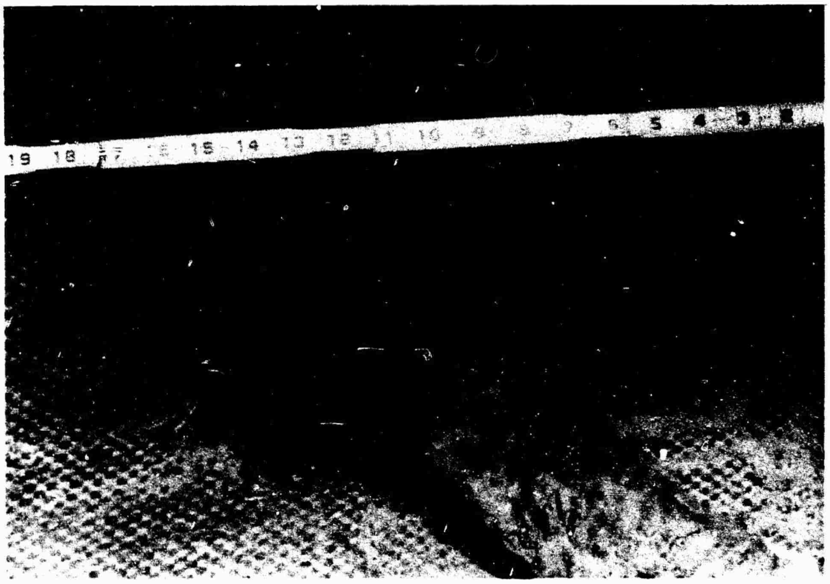
Strip drains are being installed by a sub-contractor, Geotechnics, Inc. of Bay St. Louis, Mississippi, who works off of the slag surface with a modified Koehring 266 hydraulic excavator. It has 42" wide crawler tracks and exerts 6 lb/in.^{2} ground pressure. The bucket and boom have been replaced with a 56' high set of leads and a lance. Approximately 3,000,000 lin ft of strip drains are being installed, making this project the largest such installation in this country to date. To punch through the geotextile, the rig's 15 ton static [illegible]e must be augmented by a short burst of vibratory power supplied by the excavator. Once through the geotextile, the lance is pushed easily through the soft soil and withdrawn leaving the strip drain and driving shoe behind.

Originally, a reinforcing bar was used to attach the strip drain to the end of the lance, but subsequently a flat plat driving shoe was used, see Figure 16. The resulting holes in the geotextile are shown in Figure 19. Also shown is the hole resulting from arrow-pointed shoe. A limited number of these were made for this project (see Figure 19). Typical hole sizes in the geotextile are as follows:

Table 6

Hole Sizes in Reinforcing Fabric From Strip Drain Installation

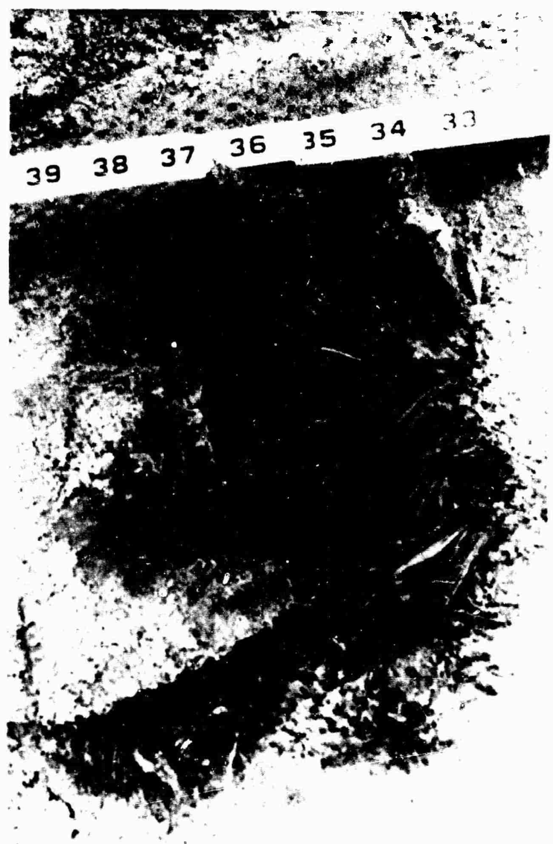
Type of Shoe	Shape of Hole	Dimensions of Hole (Length by Width)
Reinforcing Bar	Elliptic	9" x 5"
Flat Plate Shoe	Rectangular	6" x 6"
Arrow-Pointed Shoe	Rectangular	6" x 4"



(a) Using reinforcing bar

(b) Using flat plate shoe

Figure 19. Various hole sizes and shapes reinforcing fabric from strip drain installation (Continued)



(c) Using arrow pointed shoe

Figure 19. (Concluded)

After the lance is withdrawn, a laborer cuts the strip drain with a knife, doubles the loose end through the metal strap on the back of the driving shoe and staples it (often just folds it) to the new section of strip drain protruding through the lance. The laborer then hand rotates the strip drain roll to snug the driving shoe up against the bottom of the lance for the next insertion. Photographs of various phases of the installation are shown in Figure 20.

With no interruptions, one strip drain can be installed every 30 sec as the rig rotates from side to side inserting two or three rows from a single setting. The strip drain spacing is on 5' centers in a triangular pattern. Daily production (including downtime to change reels of strip drain or from major movement of the rig) ranges from 10,000 to 18,000 lin ft per 10 hour day.

The strip drain being used is Amerdrain Type 407 manufactured by The American Wick Drain Company, a division of ICE. See Table 1 for its properties and Figure 7 for its flow rate under load. Each reel consists of 1000 lin ft and weighs about 30 lb.

No information was made available to the author regarding strip drain spacing design. The specification used for the project, however, is detailed in a number of areas, e.g.

a. Strip drain must be Amerdrain Type 407, Mebradrain, Desol band drain, or as approved by the Engineer.

b. A trial or test section of the proposed installation was required to be done with the Engineer present and with his approval before job commencement.

c. The contractor must demonstrate that strip drain splicing will not decrease the strength nor reduce the flow capacity of the completed strip drain.

7.5 Surcharge Fill Details

Once the strip drains are installed, a 7' to 9' earth surcharge fill is placed above the sand drainage blanket in order to mobilize pore water pressure in the subsoil. According to the specification, this surcharge fill material is to be brought from off-site and must meet the requirements of Section 205 of the MSHA Specification for Type II Borrow Excavation. It is to be placed in three uncompacted lifts of 2' to 3' each with approximately 30 days between successive lifts. A 30 day additional time period is

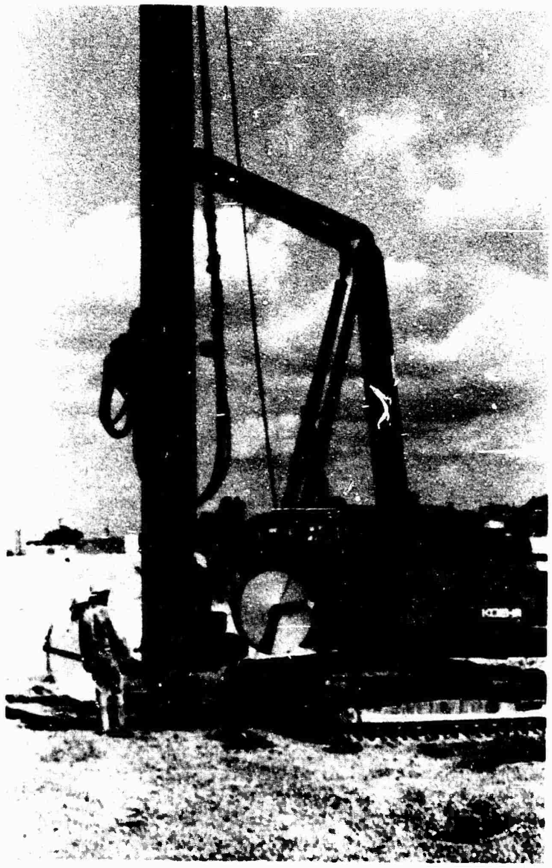


Figure 20. Photographs of strip drain installation (Continued)

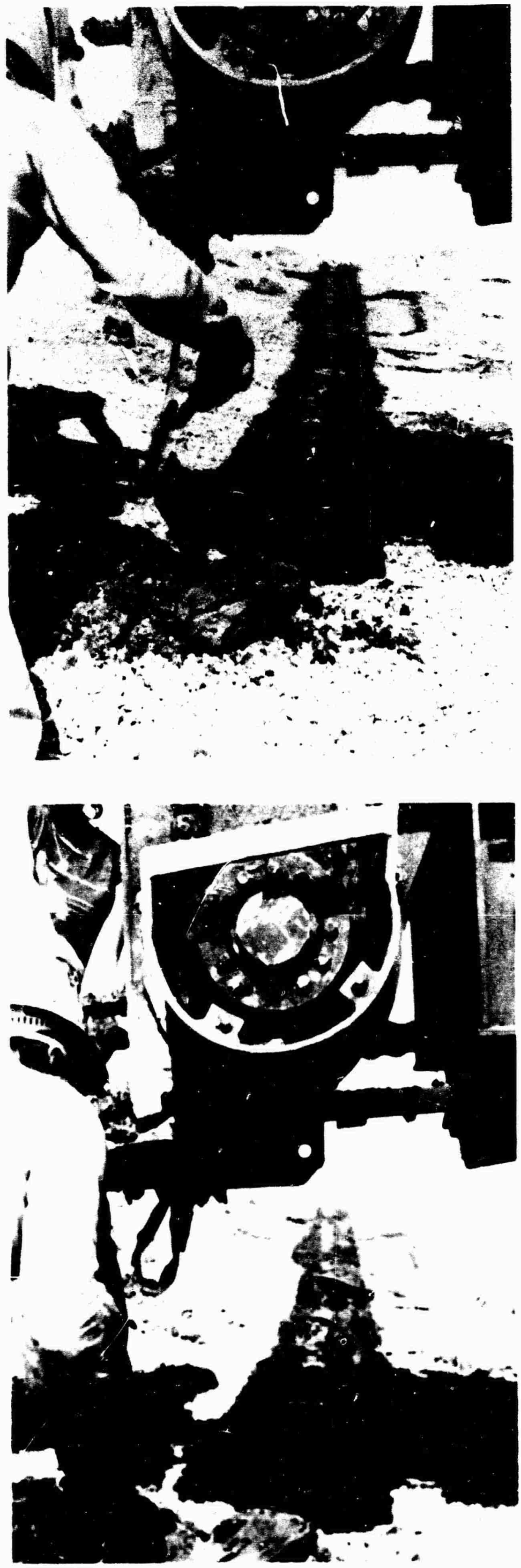


Figure 20. (Concluded)

mentioned as being a possibility if piezometer and settlement indicators indicate a slow dissipation of excess pore water pressure. Monitoring will be described in Sections 7.7, 7.8 and 7.9.

7.6 Construction Observations

During the course of this investigative report period 15 visits to the job site were made. While this is only a small fraction of the total job time, some observations of a constructive nature can be made.

a. After its placement, the fabric was left uncovered on occasions. If this cannot be controlled, either polyester must be specified or carbon black must be added to polypropylene ($\geq$4%).

b. There were unsewn gaps of 6" to 12" in the transverse seam sewing as the longitudinal seam was encountered, see Figure 21.

c. The longitudinal seams were continuous. The heavy sewing machine, held by the tractor, was well suited for the job.

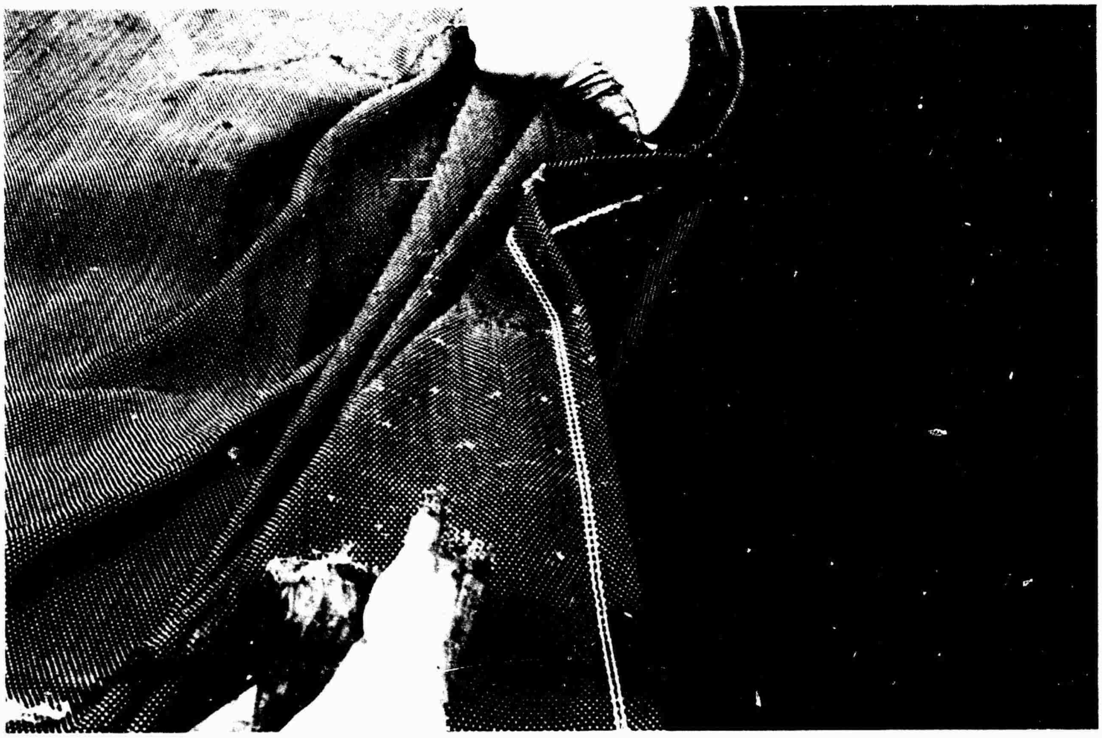
d. The fabric's flexibility for the J-stitch was appropriate and manageable. Furthermore, its size and weight were manageable for the site conditions.

e. The strip drain installation process proceeded without interruption, except when the reinforcing bar was used to hold the strip drain at the bottom of the lance. Then, the soft soil wedged itself up into the lance and prevented release of the strip drain at its intended depth. The flat plate shoe avoided this problem completely.

f. Use of the arrow pointed driving shoe also avoided the problem and allowed for lower driving forces required to penetrate the fabric. As seen previously, the added benefit of the arrow point was to minimize the hole size in the fabric, recall Tables 5 and 6.

g. The vibratory hammer on the rig was definitely effective in penetrating the fabric. With only static-down pressure the stress on the fabric would have been much more widespread, and the dip and rebound of the crane before and after fabric breakthrough would have been much more abrupt. Even with the vibratory hammer the crane rocked back and forth as penetration was effected.

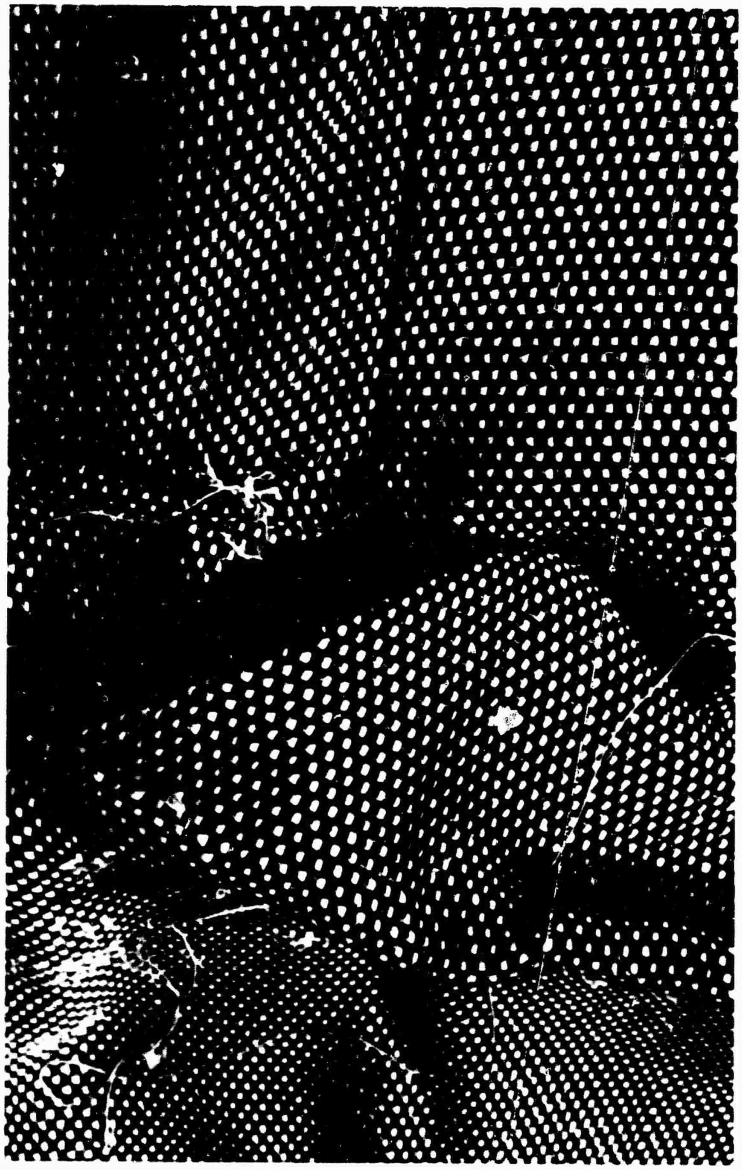
h. The random pattern of placing the strip drains was of concern. While no way to quantify its effect is known, it seems as though the work should progress systematically.



(a) Longitudinal seam

(b) Transverse seam

Figure 21. Various seams and difficulty of making complete intersections (Continued)



(c) Intersection of seams

Figure 21. (Concluded)

i. The trucks bringing fill to the site often came much too close to the uncovered fabric (urged on generally by the dozer operator so he did not have to push the sand very far). Recall the effect this has on stability, i.e., Table 4 and Section 5.10.2.

j. Spreading of sand by the small dozers was not very sequenced in a left to right (or vice versa) manner and the fabric was probably not uniformly stressed as a result.

k. The gradation of the sand drainage appeared to be finer than that called for in the specifications. This gave some concern about the ability of the expelled water to get to the proposed underdrain system.

l. The underdrain system was never installed as originally designed. This was a proper decision on the part of the resident engineer since maintaining line and grade in the sand drainage blanket was simply not possible.

m. Use of perforated 55 gal drums as sumps (placed at about 100' centers) was a very practical solution for locating pumping stations. Note Figure 22 which shows the water inflow into these drums. Large inflow occurred during even dry spells and was reasonably correlated to placement of nearby surcharge fill. Clearly, the strip drains worked as intended.

n. The 6" slag covering of the sand blanket was not very effective. This was due, in part, to the fines in the sand blanket which allowed for good trafficability by itself. However, the field personnel seemed to feel that coarse aggregate was preferable and was definitely needed.

o. Compared to the sand drainage blanket, the surcharge fill placement seemed very orderly. Long longitudinal strips were completed to the proper height using off-road trucks and medium size dozers. Road graders followed and crowned the soil at the end of the day.

7.7 Field Monitoring

Field monitoring was required according to the plans and specifications, and consists of the following devices:

a. settlement indicators, Borros type or equal (150 lin ft required), to monitor settlements at various depths of the compressible soil

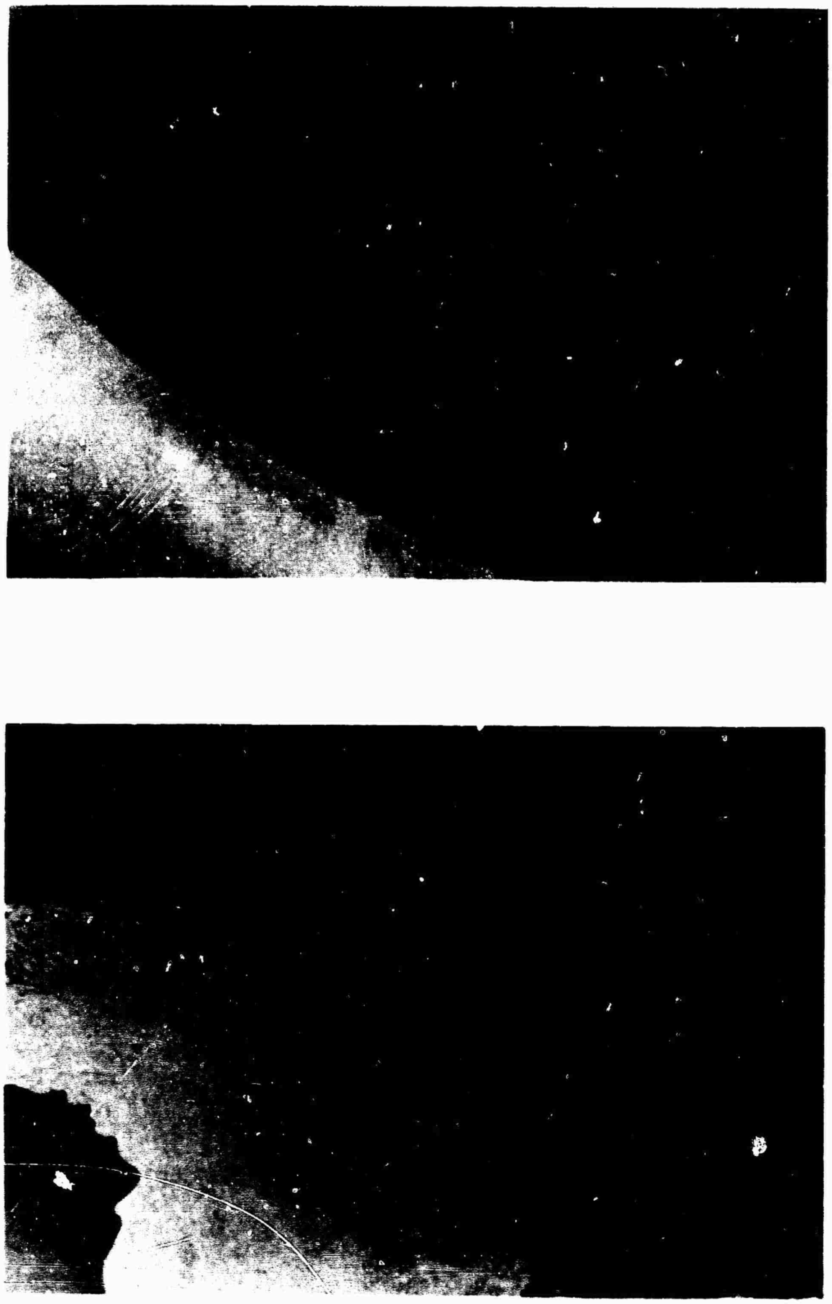


Figure 22. Photographs of water inflow for surcharged strip drains through sand drainage blanket (Continued)

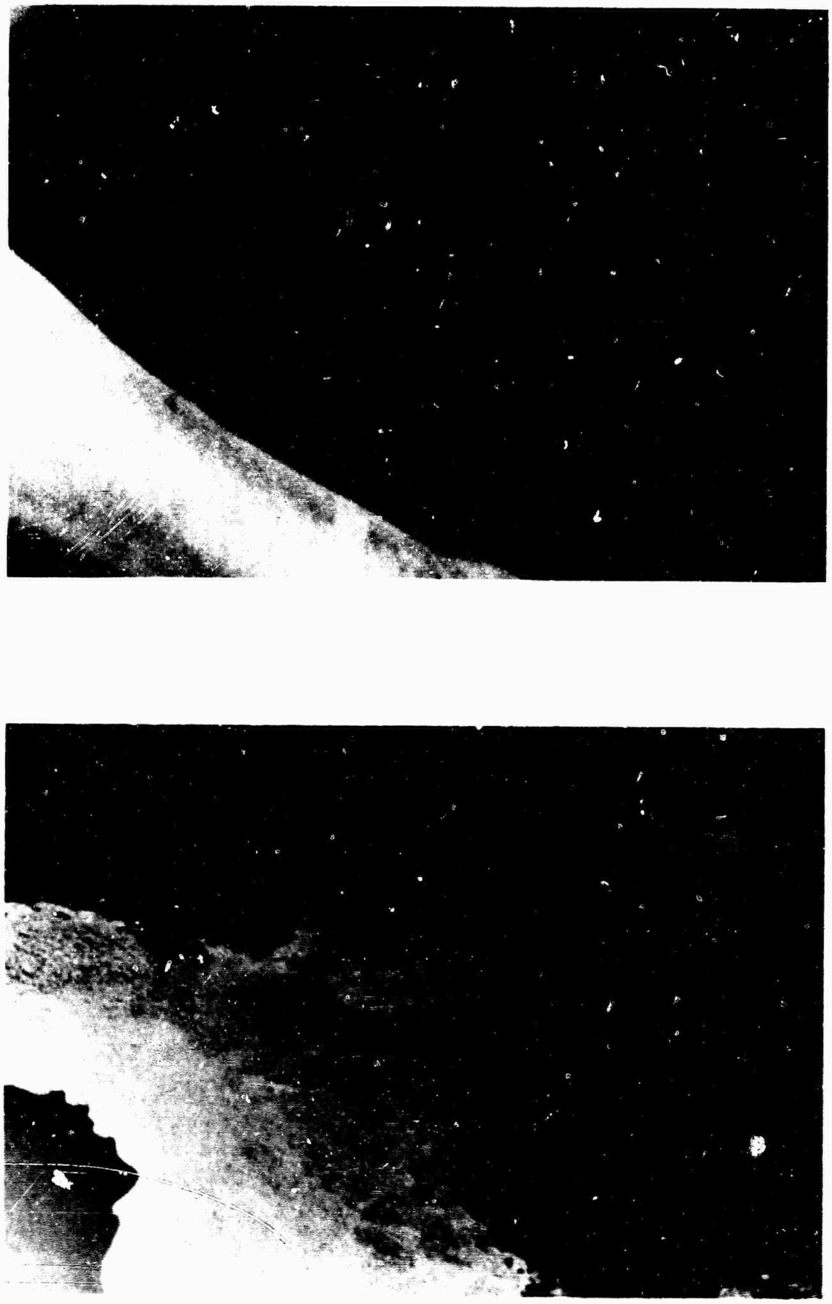


Figure 22. Photographs of water inflow for surcharged strip drains through sand drainage blanket (Continued)

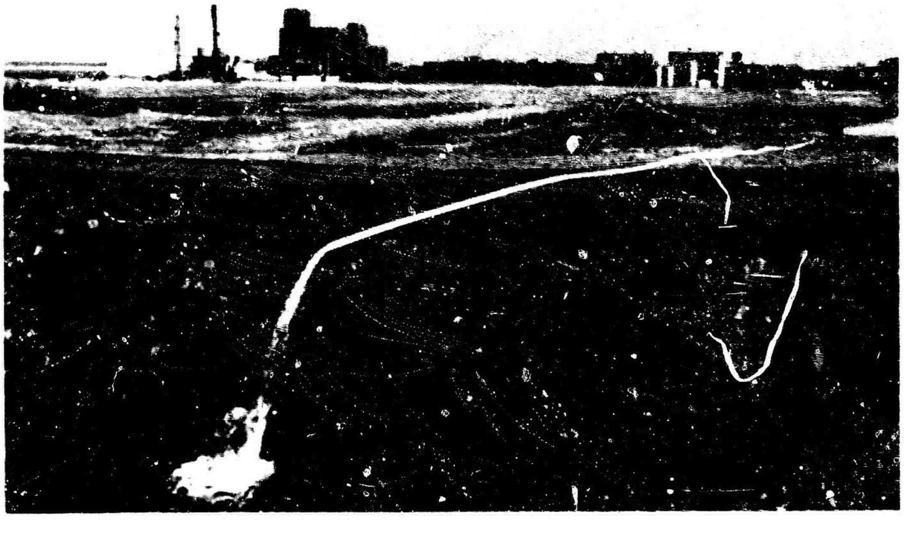


Figure 22. (Concluded)

b. settlement plates (27 required), which are placed directly on the reinforcement fabric after strip drain installation is completed (this requires excavation, placement and then backfilling of the 3' sand drainage blanket)

c. piezometers (1290 lin ft required), to be of the pneumatic type with the tip located in the compressible soil

d. slope inclinometers (400 lin ft required), to be located within the compressible soil

e. pressure cells (19 required), to be installed within the compressible soil

7.8 Field Performance

Without question, the surcharge is inducing excess pore water pressure in the subsoil, which is eventually finding its way to the sump pump areas. Water is simply pouring into these areas and is directly related to the nearness of the surcharge fill. Presumably, the route that the water is taking is into the strip drains, up into the sand drainage blanket and then laterally to the perforated 55 gal drums. Here it is pumped through hoses on the ground surface into the adjacent waterways.

The object of the field monitoring devices noted in Section 7.7, is to quantify this performance. Questions such as; what part of the subsoil is consolidating most rapidly?, what is its variation in location and depth?, are settlements and pore pressure dissipation correlated with one another?, are total stresses eventually related to surcharge loads?, are lateral deformations contributing to the vertical settlement? etc., are all capable of being answered by the job-required instruments. They are, indeed, the proper types of instruments for answering these questions.

At the time of this writing, however, results are not available for a critique of how successful they functioned, nor how effective they were in providing answers to the questions posed. Currently, strip drains are still being installed and surcharge is still being placed.

8.0 Summary and Conclusions

As a brief wrap-up of this report a few comments on its salient features are in order.

8.1 Construction on Soft Soils

A new era of soft soil construction is upon us. Proper use of geosynthetics are allowing us to work on soils heretofore impossible to handle. Previous options of excavate and replace or avoid by using deep foundations are no longer necessary. Indeed the use of high performance geosynthetics has probably saved lives[8]. Today, soils with shear strengths lower than 200 lb/ft^2 (and down to 50 lb/ft^2) are capable of being stabilized. Such soils as these have water contents significantly higher than their liquid limits, the value at which (by definition) the soil has negligible shear strength. The fact that these techniques are cost effective demands that this technology be used to its fullest extent.

8.2 Comments on Reinforcing Fabric

The key construction on very soft soils is high performance fabric (geotextiles). Tensile strengths of approximately 1000 lb/in. are being used. This allows for a single layer of fabric to be deployed and worked upon immediately. With a minimum amount of sand placed directly on the fabric, one is out of the mud and into the consolidation phase. From this point on, the situation becomes more stable rapidly.

8.3 Comments on Strip Drains

To hasten consolidation, the concept of radial flow is utilized. Rather than using the older type of sand drains, however, polymeric strip (or wick) drains are now used. Light weight, high tensile strength, light installation equipment, fast and economical; all are appropriate to describe these new materials. Indeed, strip drains have essentially caused the demise of sand drains in the space of a few short years. Strip drains have been used to depths of 40 feet and sand drains in excess of 100 feet.

8.4 Comments on Constructability

The hallmark of this system is that it can be constructed. The Corps of Engineers, the California Dept. of Transportation and now the Maryland Port Authority have clearly shown that the work is getting better, faster and less costly.

8.5 Conclusions

Unfortunately, the last aspect of this type of soft soil construction to catch-up is design. This is not unlike all of geosynthetics, however, where manufacturing and successful use have preceded design. Perhaps with projects like the one described here, and with analyses as presented in this report some of the gap has been filled.

9.0 Recommendations

Additional research and development in the use of geosynthetic materials for soft soil construction and stabilization still remains to be done. This last section outlines some directions where further inquiry should be directed.

9.1 Design Method

Here is where the majority of the work remains to be done. While the designs presented here are reasonable, they must be further tuned in order to "build-with-confidence". In the fabric area, better quantification of the effects of holes is needed along with an assessment of the sewn seam strength with and without holes. If these are indeed the limiting factors, the use of high strength fabrics may be uneconomical. In the area of strip drains a number of features need investigation, recall Table 2. A separate thrust in addition to FHWA's current efforts[9] should be initiated.

9.2 Test Methods

Much work is required in order to establish valid index and performance tests for both high strength fabrics and strip drains. It is foolish to "design by function" and then compare the required values to actual material properties which are only loosely defined or known. At the minimum we need work on the following:

- wide width tensile tests on high performance fabrics
- proper seam testing procedures
- strip drain flow information in the nondeformed and deformed (kinked) condition
- potential reinforcing benefits of strip drains after they are installed

9.3 Constructability

Perhaps the greatest need in this category is in methods to increase seam strength. A number of alternatives to sewing come to mind, e.g., mechanical, heat, ultrasonic, adhesives, etc. Better connections will better utilize the high performance reinforcing fabric's strength.

9.4 Plans and Specifications

Focus should be on performance characteristics and not on specific materials and "or equal" concepts. For designed projects such as these we must have confidence in our methods, state our requirements and let the manufacturers and contractors propose their best solutions. In this manner we will achieve the best of new and innovative solutions, at the most economical price.

10. References

1. Koerner, R. M., Construction and Foundation Methods in Geotechnical Engineering, McGraw-Hill Book Co., NY, 1984.

2. Barron, R. A., "Consolidation of Fine Grained Soils by Drain Wells," Trans. ASCE, Vol 113, 1948, pp 718-754.

3. Hansbo, S., "Consolidation of Clay by Band-Shaped Prefabricated Drains," Ground Engineering, July, 1979, pp 16-25.

4. Cortlever, N. G., "Investigation Into the Behavior of Plastic Drains in Rotterdam Harbour Sludge," Geotechnics Holland, B. V., April 1983, 16 pp.

5. van de Griend, A. A., "Deformation of Plastic Drains," Weg and Water, No. 1, Oct. 1984, Holland, 7 pp.

6. Koerner, R. M., Designing with Geosynthetics, Prentice Hall, Inc., Englewood Cliffs, NJ, 1986.

7. Nowatzki, E. A. and Pageau, S. R., "The Effect of Holes on the Tensile Performance of Geotextiles," Geotechnical Fabrics Report, Spring, 1984, pp 12-15.

8. Robison, R., "Engineering with Fabric," Civil Engineering/ASCE, Dec. 1985, pp 52-55.

9. Robison, R., Federal Highway Administration, Dept. of Transportation, Grant No. DTFH61-83-R-00059 to Haley and Aldrich, Inc., Cambridge, Mass.

10. Mitchell, J. K., Fundamentals of Soil Behavior, J. Wiley, New York, 1976.

11. Shrestha, S. C., and Bell, J. R., "Creep Behavior of Geotextiles Under Sustained Loads," Proc. 2nd Intl. Conf. on Geotextiles, Las Vegas, Nev., 1982, IFAI, pp 769-774.